Code Provisions and Practical Design Examples

of Hooked Bar Anchorage

by

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Code Provisions and Practical Design Examples

of Hooked Bar Anchorage

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Younghye Kim

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Abstract

Code Provisions and Practical Design Examples

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The University of Texas at Austin, 2009

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In structural concrete, hooked bars are used to shorten anchorage length when the requirements for straight bar anchorage cannot be provided within the available dimensions of elements. The objective of this study was to provide an overview of hooked bar anchorage. Design examples and structural details are based on Building code requirements for structural concrete (ACI 318-08) and commentary. Examples of

standard hooks in exterior beam-column joint and hooked bar anchorage details for reinforced concrete beam-SRC column joints are discussed. The general behavior of anchorage of hooked reinforcing bars is summarized from a review of previous studies. Then, design requirements for the development length of standard hook are discussed and used in an example. An example of the use of hooked bars in reinforced concrete beam-SRC column joint is provided. Four options for short development length are presented and compared: Adding more reinforcement, welding bars, confinement by steel column flanges, and anchorage by plate welded between flanges.

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CHAPTER 1
Introduction

1.1 RESEARCH MOTIVATION

Hooked bar anchorages are used when the requirements for straight bar anchorage cannot be provided within the available dimensions of elements such as exterior beam-column joints. In most cases, details for hooked bar anchorages follow general design procedures of the code provisions. To determine the development length of hooked bars (l_{dh}), Section 12.5.2 of the Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary is used. The development length of a hooked bar in Section 12.5.2 includes many of the factors based on test results (Marques and Jirsa 1972; Minor and Jirsa 1975; Pinc et al. 1977; Hamad 1990).

However, hooked bar anchorages sometime present detailing problems when hooked bar development length is greater than the available length. For example, steel-reinforced concrete (SRC) structures are defined as composite members and include structural steel members in the cross-section encased in reinforced concrete with longitudinal and transverse steel bars for reinforcing. In Korea, SRC structures are used as one of the structural systems for the top-down[1] construction method (Figure 1-1) or for seismic-resisting systems. For a reinforced concrete beam-SRC column joint, the

[1] The top-down construction method is a structural system which a high-rise superstructure and its substructure are built simultaneously after installing underground retaining walls.

reinforcement in the beam may have to be developed in a very short length less than that required by a standard hook because steel shape limits the space available for anchoring bars.

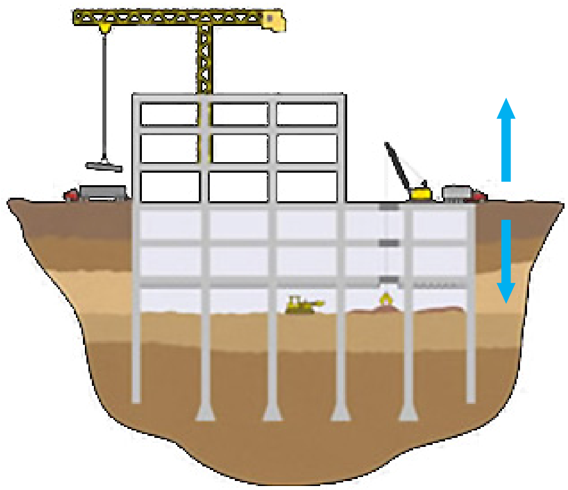


Figure 1-1: Top-down construction method (http://www.casefoundation.com)

1.2 RESEARCH OBJECTIVE

The objective of this study is the following:

1. Review the background for conventional anchorage of reinforcing bars: the mechanics of bond behavior of straight reinforcing bars;

2. Summarize previous studies on hooked bar anchorage including test results, factors affecting hooked bar anchorage, and code provisions for hooked bar anchorage in ACI 318-08;

3. Provide an overview of design examples and structural details for hooked bar anchorages that meet code provisions and an example for a hooked bar anchorages that do not meet code provisions; and

4. Propose structural details for hooked bar anchorages in reinforced concrete beam-SRC column joints with limited space available for developing bars.

1.3 SCOPE OF RESEARCH

This study mainly deals with 90° hooked bars because 90° hooked bars are typically used in the field. Material properties of reinforcement (f_y) and concrete (f'_c) are also values for typical structures.

For SRC structures in Korea, the design procedures follow provisions of "Standard for Structural Calculation of Steel-Reinforced Concrete Structures". In US, "Steel Construction Manual" of AISC (American Institute of Steel Construction Inc.) is used for design of composite axial members (Part 16: Chapter I). These provisions do not include provisions for anchorage of embedded reinforcement. For reinforced concrete beam-SRC column joints, beam design is based on the provision of KBC 2008 (Korea Building Code 2008) which is similar to ACI 318-08. Thus, the hooked bar anchorage in this study is based on the equation for a standard hook in Section 12.5.3 of ACI 318-08. In the equation for development of standard hooks, applicable factors for various field conditions will be discussed.

CHAPTER 2
Bond and Development Length of Deformed Bars

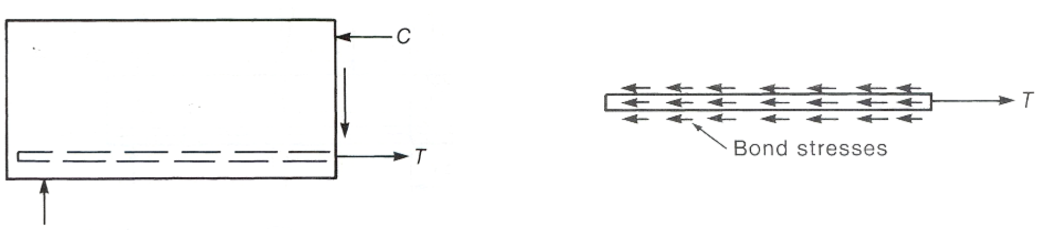
2.1 INTRODUCTION

In reinforced concrete structures, transfer of tensile forces from the reinforcing bar to the concrete is essential for satisfactory behavior. Forces are transferred through bond stresses that are dependent on not only the concrete properties but also the reinforcing bar properties. In this chapter, based on a report by ACI committee 408R, bond and development of straight reinforcing bars in tension, bond behavior and factors are discussed.

2.2 BOND MECHANISM

2.2.1 Bond Stress

Bond stress between reinforcing bar and the surrounding concrete is an important feature of the reinforced concrete behavior. As shown in Figure 2-1, tensile stress acting on the steel is transferred by bond stress to concrete.



(a) Internal forces in beam (b) Forces on reinforcing bar

Figure 2-1: Bond stress (MacGregor 2005)

Compared to crack patterns of plain reinforcing bars, cracks that form in concrete around deformed reinforcing bars have different characteristics, that is, transverse cracks propagate from the edges of the ribs are shown in Figure 2-2 (Goto 1971). Cracking influences the bond development between deformed bars and concrete.

As the reinforcing bar is loaded in tension, the tensile force is resisted entirely by the reinforcing bar at the cracks that form in the tension region of a reinforced concrete member. Between the cracks, a portion of the force is transferred to the concrete. However, as shown in Figure 2-2, the bond stress varies between cracks because of tension in concrete, and it means that yielding of the reinforcing bars starts near the crack in the reinforced concrete (Mains 1951).

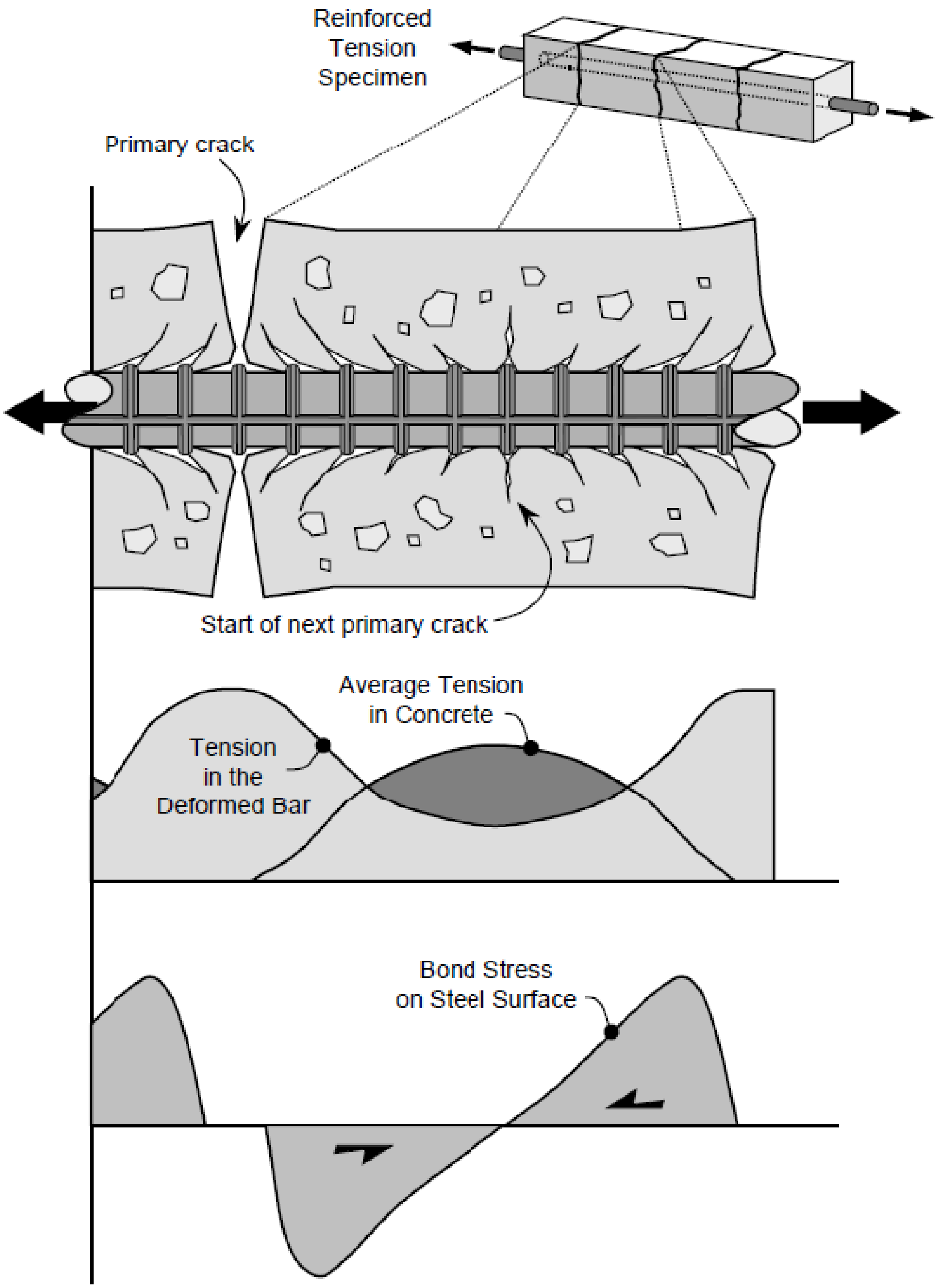
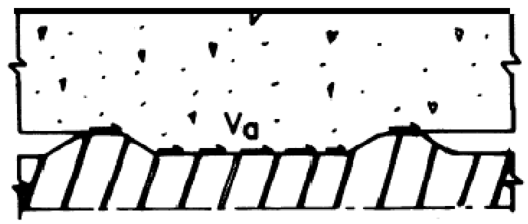
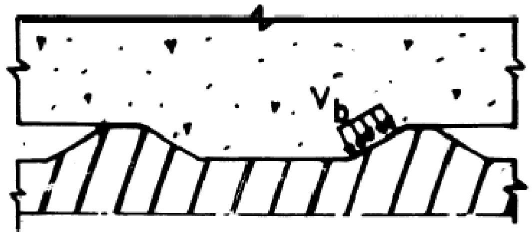


Figure 2-2: Transverse cracking at deformations (M. K. Thompson 2002)

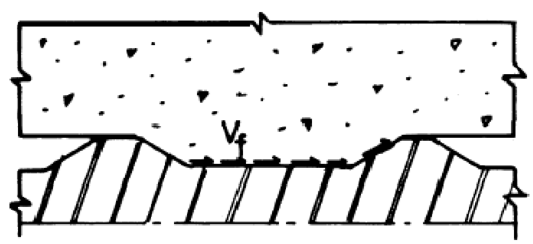
For deformed reinforcing bars, bond stress is developed by a combination of bearing forces on the rib of the bar, adhesion force, and friction force along the surface of the bar (Figure 2-3).



(a) Adhesion



(b) Bearing



(c) Friction

Figure 2-3: Mechanic of bond stress on concrete (ACI Committee 408 2003)

Bearing stresses which act on the face of the rib with an angle, θ, can be considered as resultants of parallel and perpendicular forces (Figure 2-4). These forces act on the concrete in the opposite direction. Of the two components, the parallel forces resist tensile force in the reinforcing bar. The perpendicular forces act outward forces from the bar and lead to splitting of the concrete. These radial splitting stresses on the bar produce ring tension in the surrounding concrete.

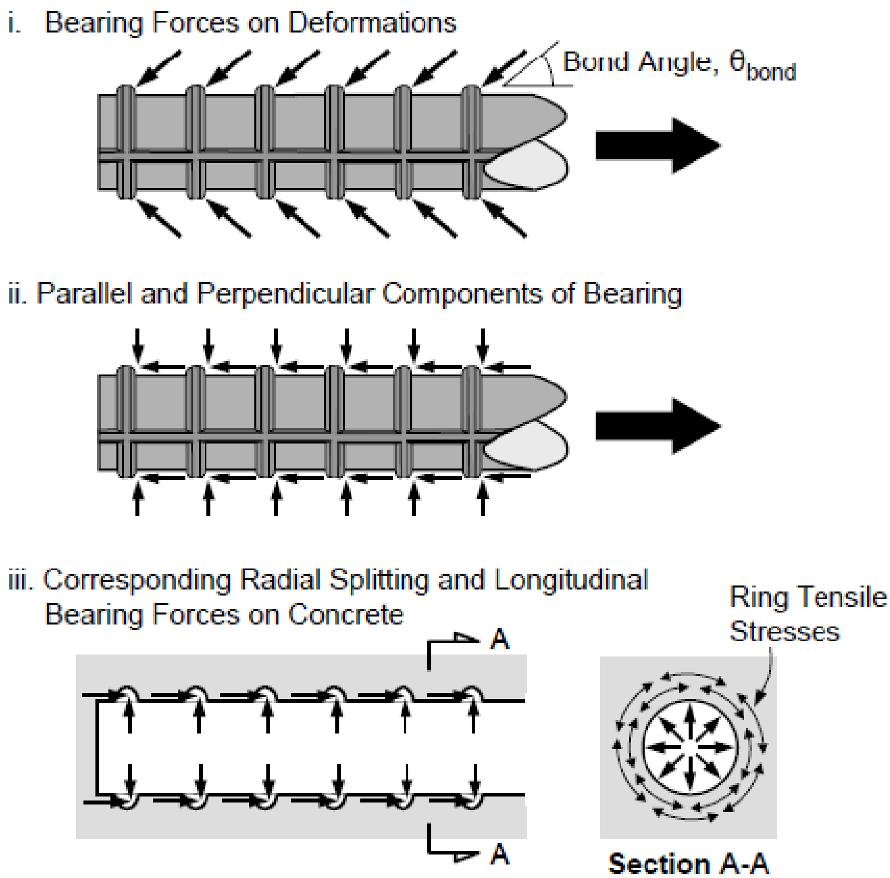


Figure 2-4: Bond and splitting components of rib bearing stresses (M. K. Thompson 2002)

Eventually, when the tensile capacity of the surrounding concrete can no longer resist the radial splitting stresses, splitting cracks begin to propagate from the ribs of the bar to surface of concrete. Once these cracks develop, the bond stress drops rapidly.

2.2.2 Failure Mode

Two failure modes can be observed. The first type of failure is a “pullout” failure by shearing along the ribs of bar. The second failure mode is a “splitting” failure which is splitting of the concrete cover when the concrete cover is small or bars are closely spaced.

2.2.2.1 Pullout failure

When the longitudinal bond stresses exceed the shear strength of the concrete between ribs of the bar, the concrete fails by shear and the reinforcing bars can be pulled out. For this mode of failure, the concrete cover and bar spacing of transverse reinforcement must be sufficient. The concrete strength, and the pattern and geometry of the deformations on the bars affect pullout failure.

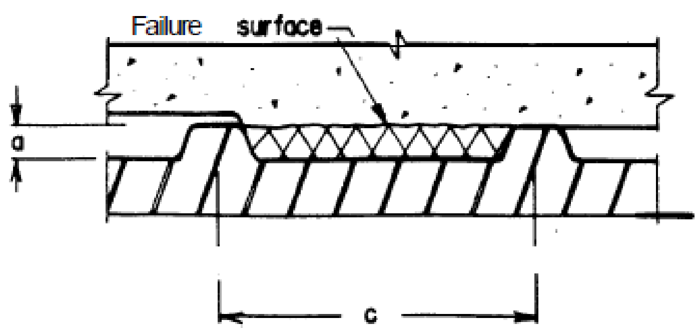
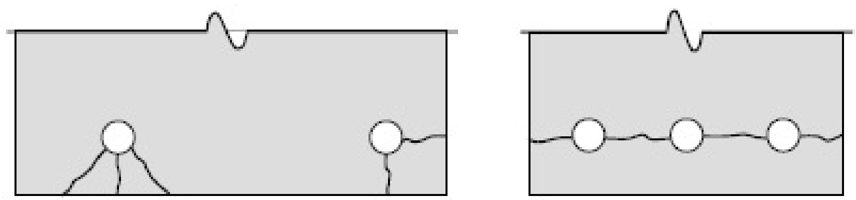


Figure 2-5: Pullout failure (ACI Committee 408 2003)

2.2.2.2 Splitting failure

Splitting failure occurs if the concrete cover or confinement is not adequate to obtain a pullout failure. Splitting failure is result of radial tensile stresses on concrete, and the cracks will propagate from the bar through the cover or to a crack from an adjacent bar. As a result, the concrete cover will spall and the bond stress will be lost.



(a) V-notch (b) Face and side split (c) Side split

Figure 2-6: Splitting crack patterns (M. K. Thompson 2002)

Figure 2-6 exhibits crack patterns of splitting failure on the reinforcing concrete member. The splitting cracks tend to develop along the shortest distance between a bar and the surface or between two bars. In general, the type of splitting failure is governed by spacing between reinforcing bars, cover dimensions, the tensile strength of the concrete, and the average bond stress.

2.3 FACTORS AFFECTING BOND

Bond behavior is influenced by geometric factors and material properties. These factors are discussed in the following categories: structural characteristics, bar properties, and concrete properties.

2.3.1 Structural characteristics

2.3.1.1 Concrete cover and bar spacing

The crack patterns of splitting failures depend on the concrete cover (c_{c1}), the concrete side cover (c_{c2}), and the bar clear spacing (s_{cb}) (Orangun et al. 1977). The factors are shown in Figure 2-7.

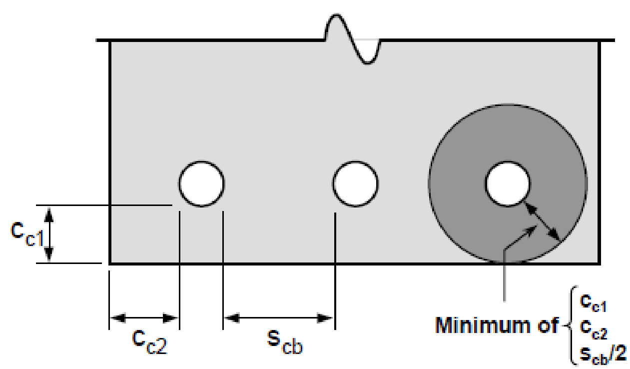
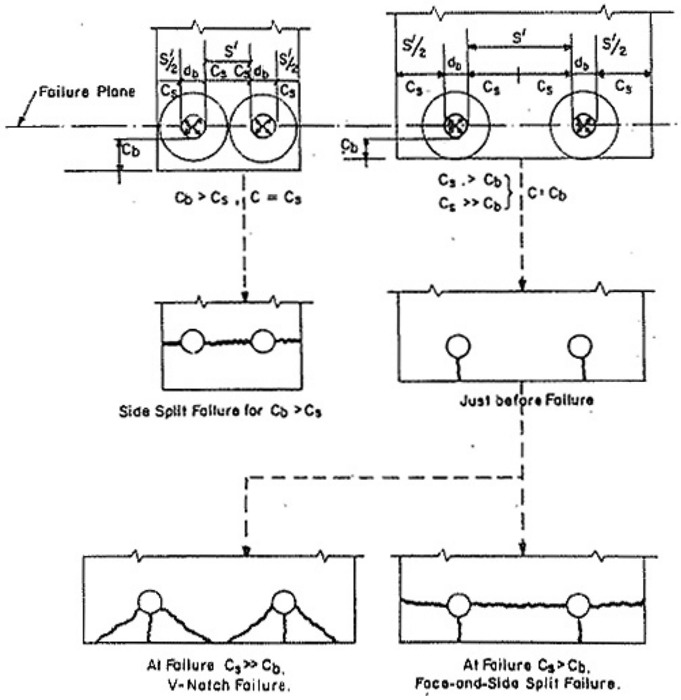


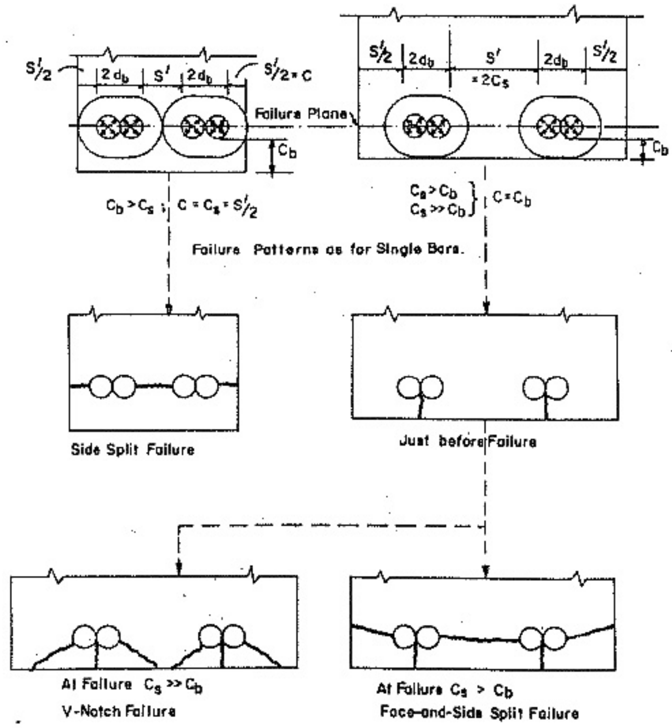
Figure 2-7: Factors for bond crack (M. K. Thompson 2002)

As mentioned at Section 2.2.2, the failure modes in bond are related to the concrete cover and bar clear spacing. Pullout failure occurs for large cover and bar spacing, whereas, for smaller cover and bar spacing, a splitting failure occurs. The splitting failure mode is generally expected in most structural members.

Figure 2-8 shows the splitting crack patterns at single bars and spliced bars.



(a) Single bars



(b) Spliced bars

Figure 2-8: Splitting failure (Orangun et al. 1977)

When splitting failure occurs, splitting crack patterns are defined by relationship between the concrete cover (c_{c1}), the concrete side cover (c_{c2}), and the bar clear spacing (s_{cb}). When c_{c1} is smaller than c_{c2} and s_{cb}, the splitting crack occurs through the cover to the free surface. When c_{c2} or s_{cb} is smaller than c_{c1}, the splitting crack forms through the side cover or between the reinforcing bars, respectively.

2.3.1.2 Development and splice length

The development or splice length of a reinforcing bar affects bond capacity. In addition, previous research showed that ribs of deformed bar bearing against the concrete result in higher bond strength than for plain bars.

When reinforcing bars are lapped spliced, the tensile force in one bar is transferred to the adjacent bar through the concrete between the bars. The radial splitting forces around spliced bars form an oval shaped tensile zone, and these forces cause splitting cracks along the bars that are similar to the single bar.

For spliced bars, splitting starts at the ends of the splice, moving towards the center. Figure 2-9 shows the zone of tension stresses and the typical splitting crack patterns in spliced bars.

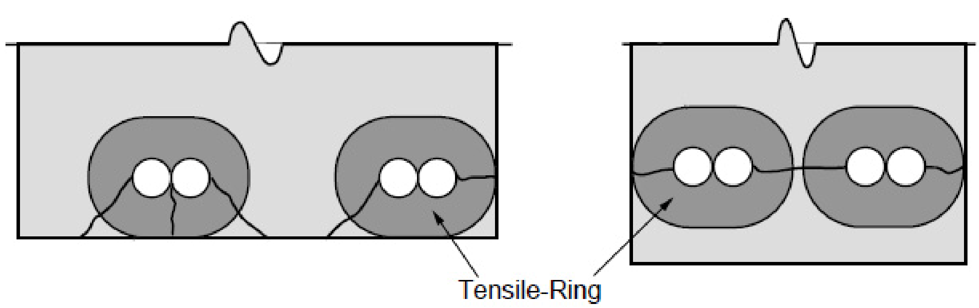


Figure 2-9: Zone of tension on spliced bars (M. K. Thompson 2002)

2.3.1.3 Transverse reinforcement

Spirals, transverse ties, and stirrups are examples of transverse reinforcement for confinement. The opening of the splitting cracks is restricted by the transverse reinforcement in the developed or lapped splice region of reinforcing bars, and such confinement increases the bond capacity. Although an increase in transverse reinforcement results in an increase in bond force, excessive transverse reinforcement causes a transition from a splitting failure to a pullout failure (Orangun et al. 1977).

Figure 2-10 shows the function of transverse reinforcement across a splitting crack. The transverse reinforcement does not resist tensile splitting stresses until the splitting cracks intersect the transverse reinforcement. The splitting cracks cross the transverse reinforcement, and then the cracks tend to taper off. Therefore, when the transverse reinforcement is placed close to a longitudinal bar, the splitting crack is controlled more effectively.

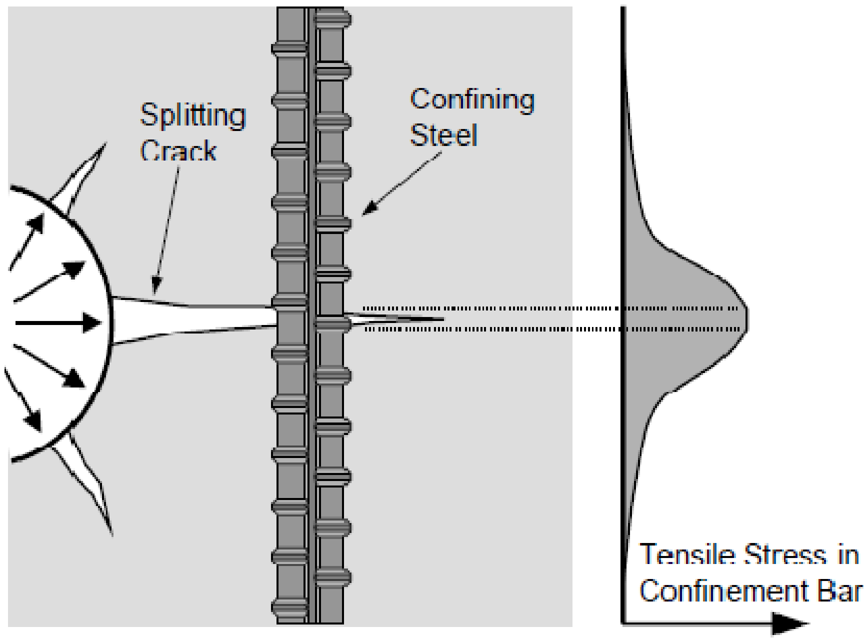


Figure 2-10: Confinement steel in the vicinity of a splitting crack
(M. K. Thompson 2002)

2.3.1.4 Bar casting position

Top reinforcement is defined as horizontal reinforcement with more than 12 in. of fresh concrete cast in the member below the development length or splice. During the placement and vibration of fresh concrete, water and air pockets in the concrete move upward, while heavy components such as aggregates may settle to the bottom of members (Figure 2-11). As the result, the light components are trapped on the underside of reinforcing bars, creating a weaker interface on the bottom of the bar that leads to a decrease in bond capacity. Thus, a 30% increase in development length is specified in the ACI code for top reinforcement (ACI318-08 Section 12.2.4).

Previous studies shows that top-cast bars have lower bond strengths than bars cast lower (Clark 1946; Jirsa and Breen 1981). A report by ACI committee 408 indicates that the lower bond strength of top-cast bars may be explained as follows: *the greater the depth of concrete below a reinforcing bar, the greater will be the settlement and accumulation of bleed water at the bar, because there is more concrete beneath the bar to settle and bleed. The effects of settlement and bleeding on the concrete around a bar are aggravated by increased concrete slump and decreased cover above the bar*.

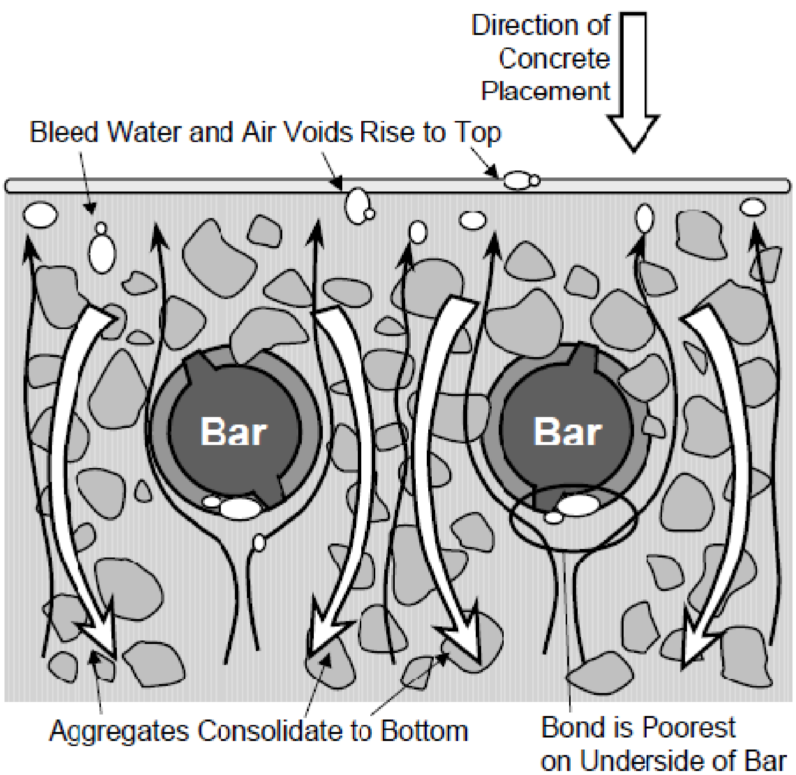


Figure 2-11: Top cast bar effect (M. K. Thompson 2002)

2.3.2 Bar properties

2.3.2.1 Bar size

Larger bars require larger forces when the bars fail by the splitting or pullout failure. This result means that the bar area is also a factor along with concrete cover, bar spacing, and bonded length (Darwin et al. 1992; Darwin et al. 1996; Orangun et al. 1977). When considering bond stress, using a larger number of small bars is more effective than using a smaller number of large bars due to an increase in perimeter of bars for the same bar area (Ferguson 1977).

2.3.2.2 Bar geometry

Figure 2-12 shows various the geometries of reinforcing bars.

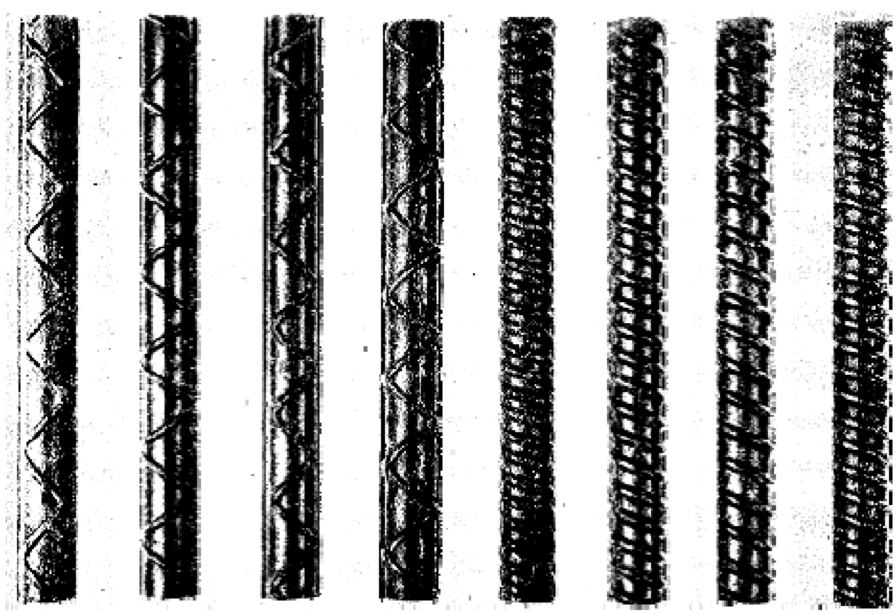


Figure 2-12: Types of bar geometry (Clark 1946)

Previous studies indicate that deformed bars can develop higher bond strength than plain bars, because as slip continued, the ribs of deformed bars increase bond resistance by bearing on the concrete.

Thompson et al. describe mechanics of ribs in a literature review for headed bars as the followings: *as a rib begins to bear on the concrete a wedge of crushed paste is formed in front of the rib (Figure 2-13). This wedge acts to change the effective face angle of the rib. Thus, the bond angle, θ, tends to change as a reinforcing bar acquires load. The effect of this is that radial splitting stresses tend to increase at a rate greater than the longitudinal bond stresses as tensile load in the reinforcing bar rises. Furthermore, efforts to reduce splitting stresses in a reinforcing bar by fabricating a steep rib angle into the bars tend to be unsuccessful because the formation of the concrete wedges neutralizes the effect of the different rib angles. Rib bearing area can be increased by manipulating one or both of two geometric parameters: the height of the ribs or the spacing of the ribs. Rib bearing area is generally referred to by the ratio of rib bearing area to shearing area of the concrete keys between successive ribs. This ratio is referred to as the relative rib area,* R_r. *The effect of the relative rib area has been studied since the earliest research on bond. Previous studies (Abrams December, 1913; Clark 1946; Clark 1949) indicate that bond stiffness was enhanced by increases in relative rib area*.

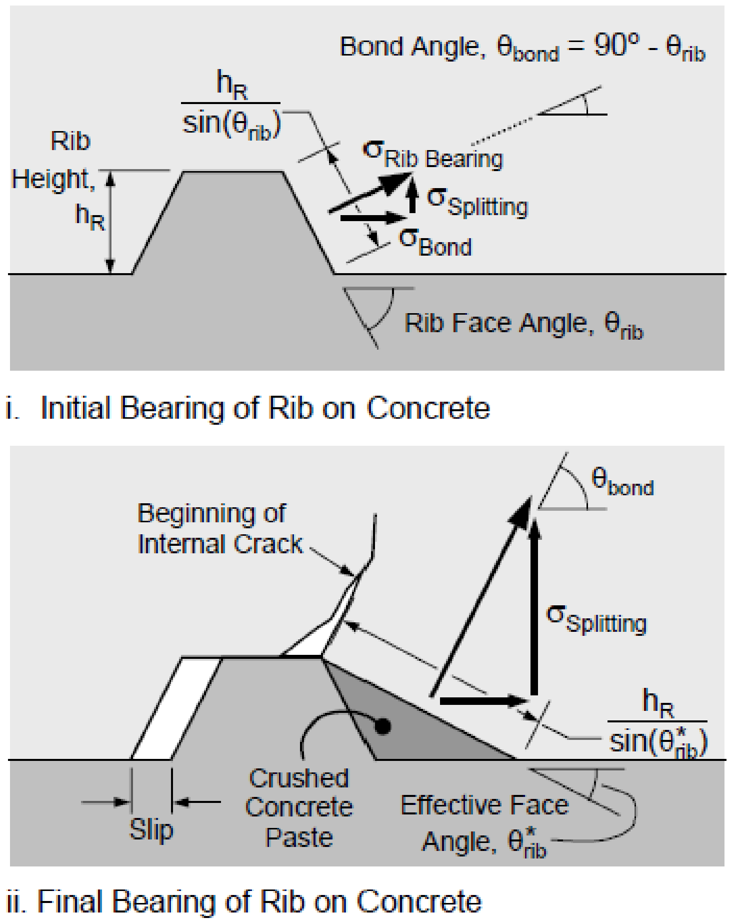


Figure 2-13: Mechanics of rib bearing on concrete (M. K. Thompson 2002)

2.3.2.3 Steel yield strength

In general, bond stress is affected by the steel strength, that is, the required bond strength reduces when lower strength steel used. However, using confinement reinforcement may be required to increase bond stress when using high strength steel (Darwin et al. 1996; Zuo and Darwin 1998; Zuo and Darwin 2000).

2.3.2.4 Bar surface condition

2.3.2.4.1 Bar cleanliness

Bond strength can be increased or decreased by friction between the reinforcing bar and concrete. Thus, bar surface condition plays an important role in bond strength. The ACI code requires that reinforcement in structural members is free of mud, oil, and other nonmetallic coatings to increase bond strength (ACI 318-08 Section 7.4). However, previous research shows that reinforcing bars with limited rust have slightly increased bond capacity because of an increase of friction between reinforcing bars and concrete.

2.3.2.4.2 Epoxy-coated bars

Epoxy-coatings are used to prevent corrosion of reinforcing bars which causes deterioration of reinforced concrete structures (Figure 2-14). Compared with uncoated bars, bond strength of epoxy-coated bars is lower due to a decrease in friction and bearing capacity on concrete. Tests of epoxy-coated bars show that the epoxy-coating thickness has little effect on the reduction in bond strength for larger bars, while bond strength for smaller bar decreases with increasing coating thickness (Darwin and Graham 1993; Hamad et al. 1993; Treece and Jirsa 1989).

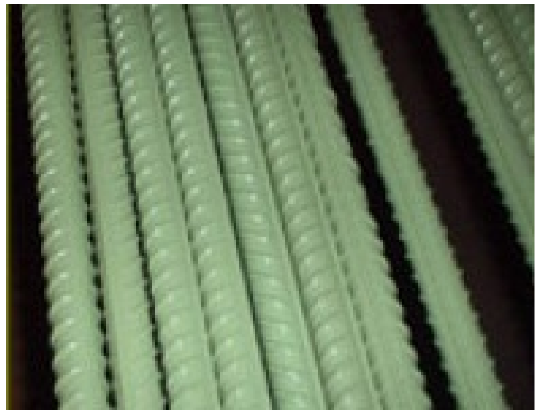


Figure 2-14: Epoxy-coated bars (www.alibaba.com)

2.3.3 Concrete properties

2.3.3.1 Compressive strength

The effect of concrete properties on bond strength is represented by the square root of the compressive strength ($\sqrt{f'_c}$) (Darwin et al. 1992; Esfahani and Rangan 1998a; Esfahani and Rangan 1998b; Orangun et al. 1977; Tepfers 1973). The ACI code requires that the value of $\sqrt{f'_c}$ for development and splices of reinforcement must not exceed 100 psi (ACI 318-08 Section 12.1.2). The reason is that for high strength concrete, the average bond strength at failure based on $\sqrt{f'_c}$ decreases with an increase in compressive strength. Recent research shows that $\sqrt[3]{f'_c}$ may represent the effect of concrete strength on bond better than $\sqrt{f'_c}$ but the ACI code has not adopted that relationship. The rate of decrease depends on splice length. Previous studies indicate that the bearing capacity of concrete (related to f'_c) is related more strongly with compressive strength than with tensile strength (related to $\sqrt{f'_c}$). The higher bearing capacity that the high-strength concrete provides delays crushing of the concrete in front of the ribs, and reduces local slip on the reinforcing bar. Thus, fewer ribs transfer load which results in a splitting failure (Azizinamini et al. 1995; Azizinamini et al. 1993; Hamad and Itani 1998; Zuo and Darwin 1998; Zuo and Darwin 2000).

2.3.3.2 Aggregates

The use of higher strength aggregate increases the bond strength (Zuo and Darwin 1998; Zuo and Darwin 2000) and delays splitting failure (Barhan and Darwin 1999; Kozul and Darwin 1997). Splitting failure is dependent on the tensile strength of concrete. Thus, the tensile properties of the aggregate affect bond strength.

2.3.3.3 Lightweight concrete

Lightweight concretes are produced by using lightweight aggregates such as slags, fly ashes or expanded clays. In general, the lightweight concretes are weaker in tension and shear than normal weight concretes. Therefore, lightweight concrete has lower bond capacity because of weaker tensile properties. As a result, the ACI code requires a factor 1.3 for determining development length in lightweight concrete (ACI 318-08 Section 12.2.4).

CHAPTER 3

Summary of Hooked Bar Anchorage

3.1 STANDARD HOOKS

Standard hooks are used when the available length is less than that required for a straight bar anchorage. The details of 90° and 180° standard hooks in Section 12.5 of ACI 318-08 are shown in Figure 3-1.

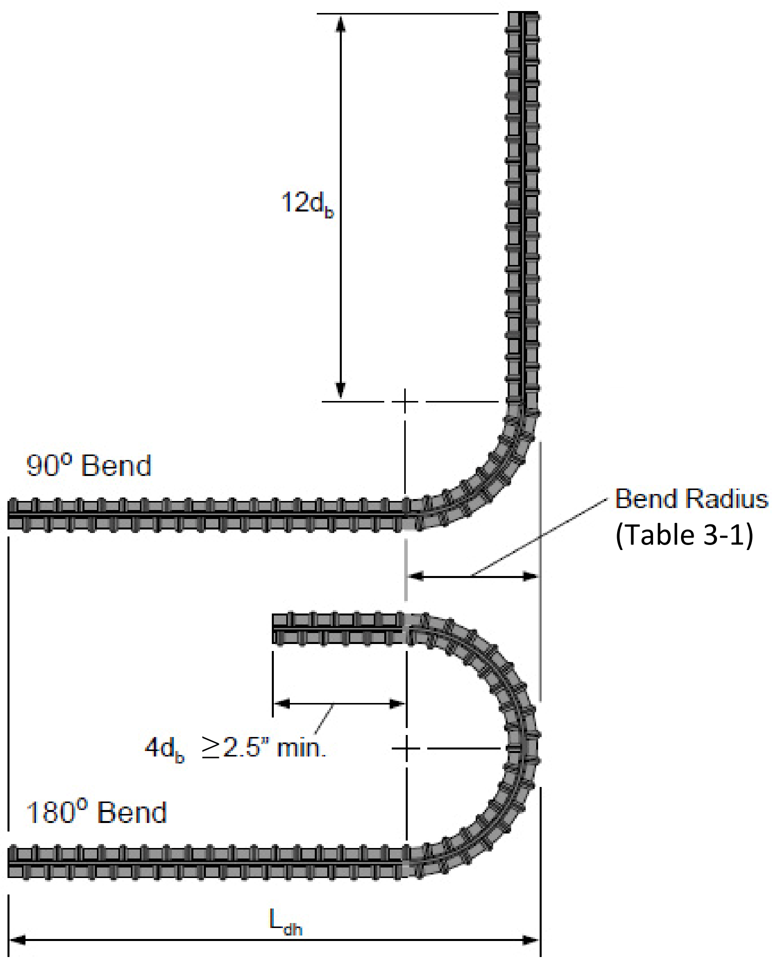


Figure 3-1: Standard hook dimensions (M. K. Thompson 2002)

Based on Section 7.2 of ACI 318-08, Table 3-1 shows minimum inside diameters of bend on standard hooked bars which are based on flexural strains in the reinforcement not producing fracture of the steel.

Table 3-1: Radius of bend

Bar size	Bend radius
No.3 through No.8	$4d_b$
No.9, No. 10, and No.11	$5d_b$
No. 14, and No.18	$6d_b$

The tension force applied to a hooked bar is resisted by a combination of bond on the surface of the bar and by the bearing on the concrete inside the hook (Figure 3-2).

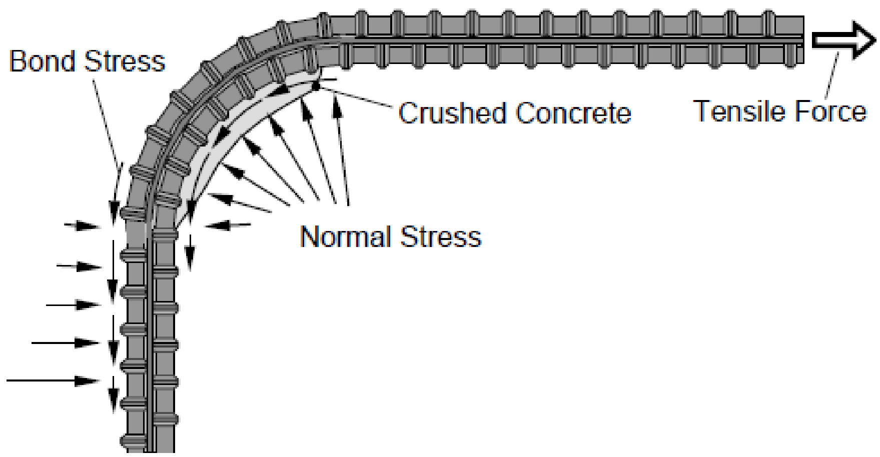


Figure 3-2: Stress transfer in a hooked bar (M. K. Thompson 2002)

For a 90° hooked bar, as the tensile force reaches full development, the inside of the bend bears on the concrete. The tail of the 90° hook tends to straighten and compressive stresses on the outside of the tail resist the prying action of the tail extension.

When the hooked bar anchorage fails, crushing of the concrete inside the radius always occurs. If the clear cover normal to the plane of the hooked is not sufficient, the side cover will spall out as lateral forces in the area where crushing occurs react against the cover. Figure 3-3 shows failure of side spalling of concrete cover.

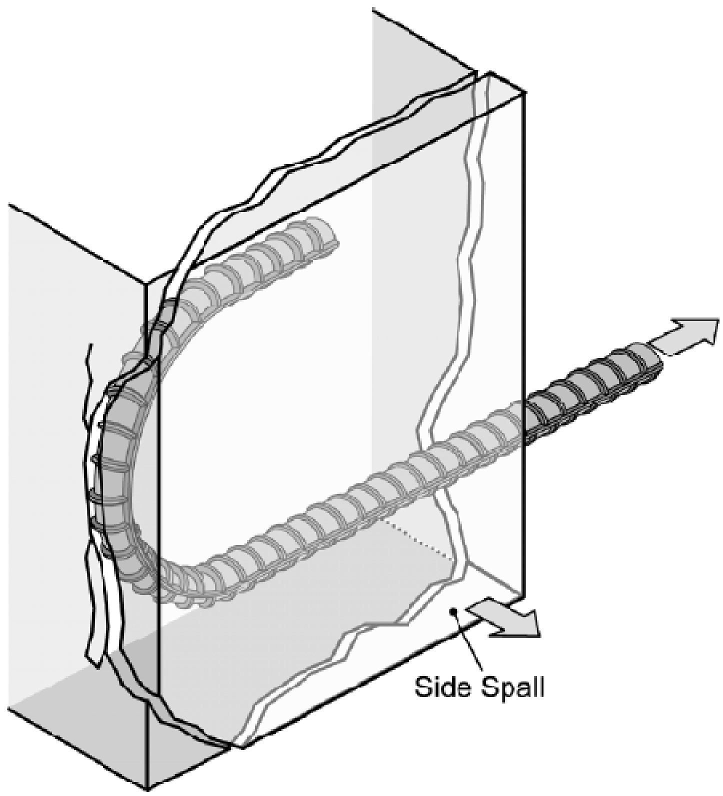
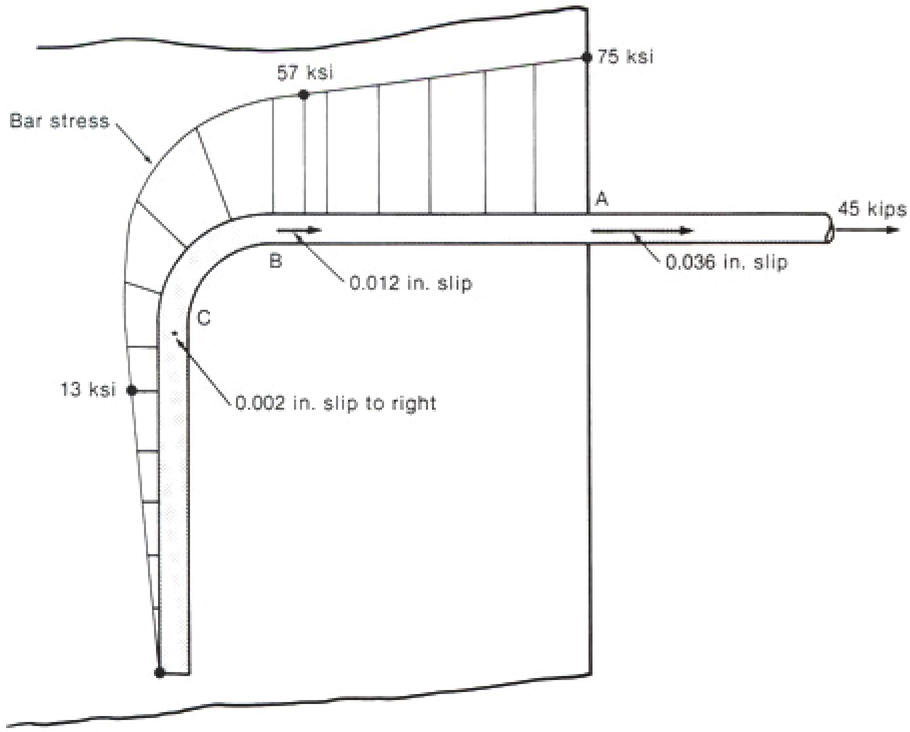
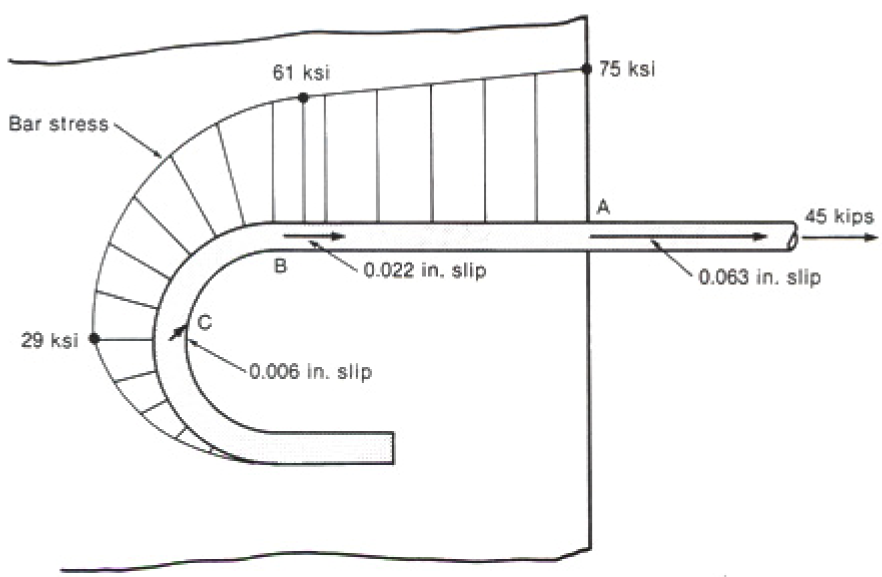


Figure 3-3: Side spall failure of hooked bar (M. K. Thompson 2002)

As shown in Figure 3-4, the angle of bend affects the amount of slip. The stresses decrease along the bend. Comparing the behavior of a 90° hooked bar with a 180° hooked bar, at point A the slip on the180° hooked bar is larger (1.75 times) than that of the 90° hooked bar.



(a) Stresses and slip - 90° standard hook



(b) Stresses and slip - 180° standard hook

Figure 3-4: Stresses and slip of #7 standard hook bar (MacGregor 2005)

The ACI code provides design equations for development length and standard details for 90° and 180° hooked bars in Section 12.5. The development length of hooked bars is given by Eq.3-1:

$$l_{dh} = \frac{0.02\psi_e f_y}{\lambda\sqrt{f'_c}} d_b \geq max\left(8d_{b,}\, 6\, in.\right) \qquad \text{(Eq. 3-1)}$$

The factors in the equation are as follows (Section 12.5.2):

ψ_e = *factor used to modify development length based on reinforcement coating*

(ψ_e= 1.0 for uncoated reinforcement, ψ_e= 1.2 for epoxy-coated reinforcement)

λ = *modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normalweight concrete of the same compressive strength*

(λ = 1.0 for normalweight concrete, λ = 0.75 for lightweight concrete)

Length l_{dh} in 12.5.2 shall be permitted to be multiplied by the following applicable factors (Section 12.5.3):

(a) For #11 and smaller hooks with side cover ≥ 2.5", and 90° hook with cover on bar extension beyond hook ≥ 2.5"…………………………………………. 0.7

(b) For 90° hooks of #11 and smaller bars either enclosed within ties or stirrups perpendicular to the bar being developed, spaced ≤ $3d_b$along l_{dh}; or enclosed

within ties or stirrups parallel to the bar being developed, spaced $\leq 3d_b$ *along the length of tail extension of the hook plus bend* ………………………… *0.8*

(c) For 180° hooks of #11 and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced $\leq 3\,d_b$ *along* l_{dh} ……………………………………………………………………………… *0.8*

(d) Where anchorage or development for f_y *is not specifically required, reinforcement in excess of that required by analysis* … *(*A_s *required)/ (*A_s *provided)*

Equation 3-1 does not include a factor for top-cast bars because hooked bar anchorages develop most of their strength by direct bearing, and not by bond along the surface area of the bar. Based on the equation, development length of hooked bars is shorter than straight bars. Figure 3-5 shows a comparison of development and splice lengths for No.8 hooked and straight bars versus concrete compressive strength.

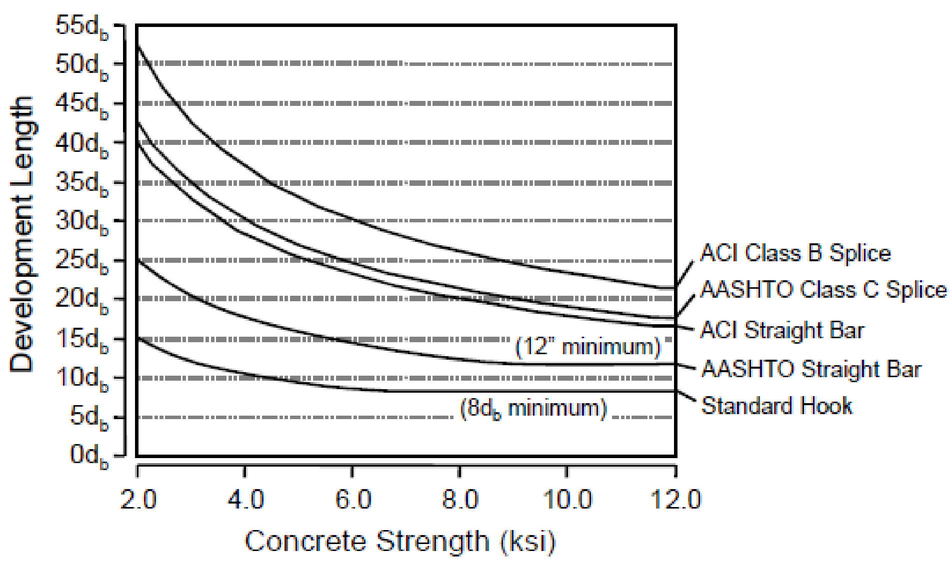


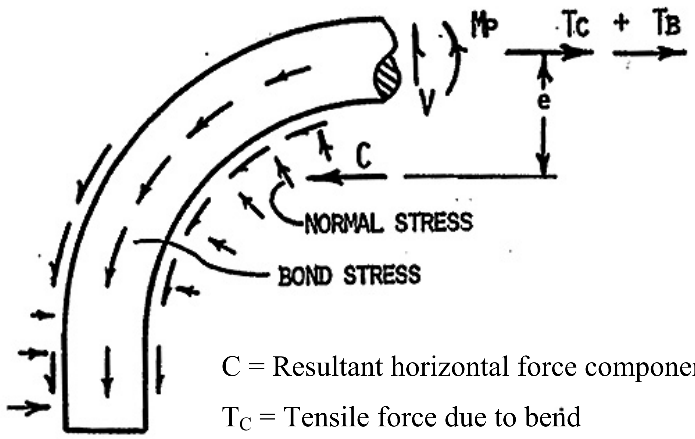
Figure 3-5: Development lengths of standard hooks and straight bars

(M. K. Thompson 2002)

3.2 SUMMARY OF PREVIOUS RESEARCH ON HOOKED BARS

3.2.1 Minor and Jirsa – A study of bent bar anchorages in concrete (Minor and Jirsa 1971; Minor and Jirsa 1975)

Pullout tests were conducted in order to examine some factors which influence the anchorage capacities of bent bar. Eighty specimens contain different geometric configurations such as bond length, angle included in the bend, inside radius of bend, and bar diameter on the deformation and strength of hooked bar anchorages. The test results are shown as load-slip curves compared the behavior of different bar geometries. Figure 3-6 shows force mechanism on bend bar when the tension is applied.



C = Resultant horizontal force component of normal stresses

T_C = Tensile force due to bend

T_B = Tensile force due to bond stresses

Figure 3-6: Forces on bent bar (Minor and Jirsa 1971)

3.2.1.1 Angle of bend

The test results show that as the angle of bend increases, the initial stiffness or the initial slope of the load-slip curve tends to decreases. In addition, Figures 3-7 and 3-8 show that an increase in the angle of bend results in larger values of slip at the same bar stresses. The mechanism of slip on bent bar is shown in Figure 3-9.

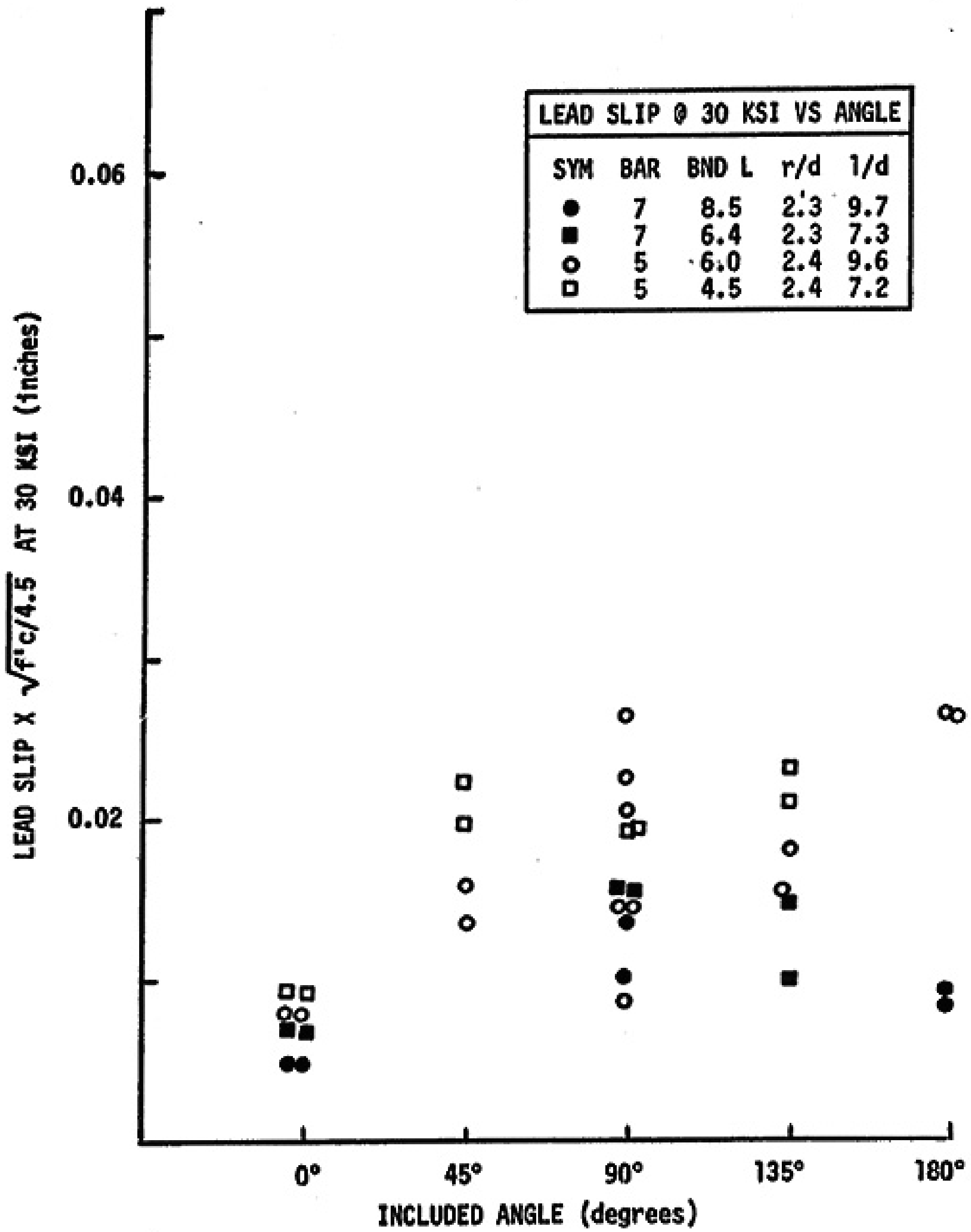


Figure 3-7: Effect of included angle on slip at 30 ksi (Minor and Jirsa 1971)

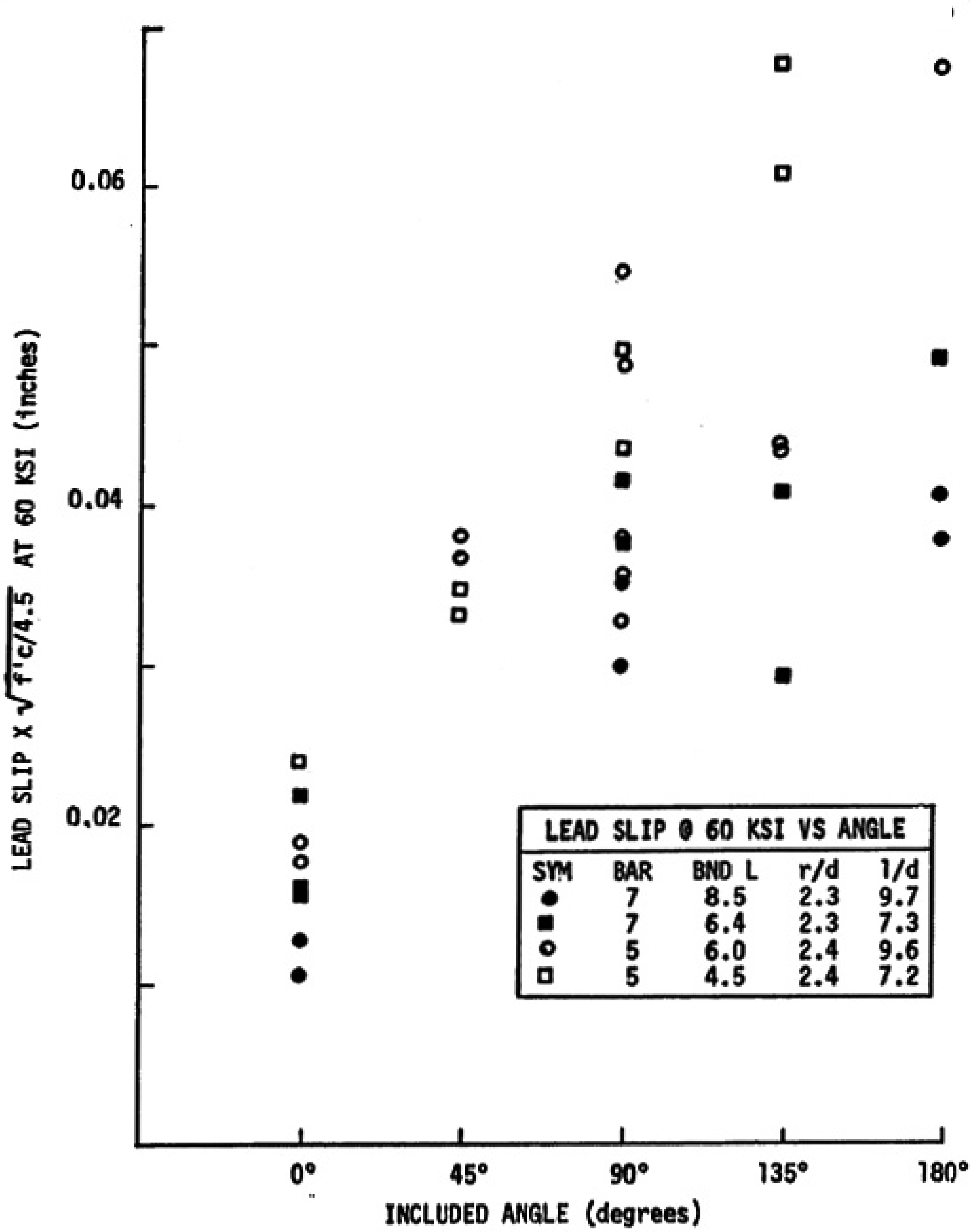


Figure 3-8: Effect of included angle on slip at 60 ksi (Minor and Jirsa 1971)

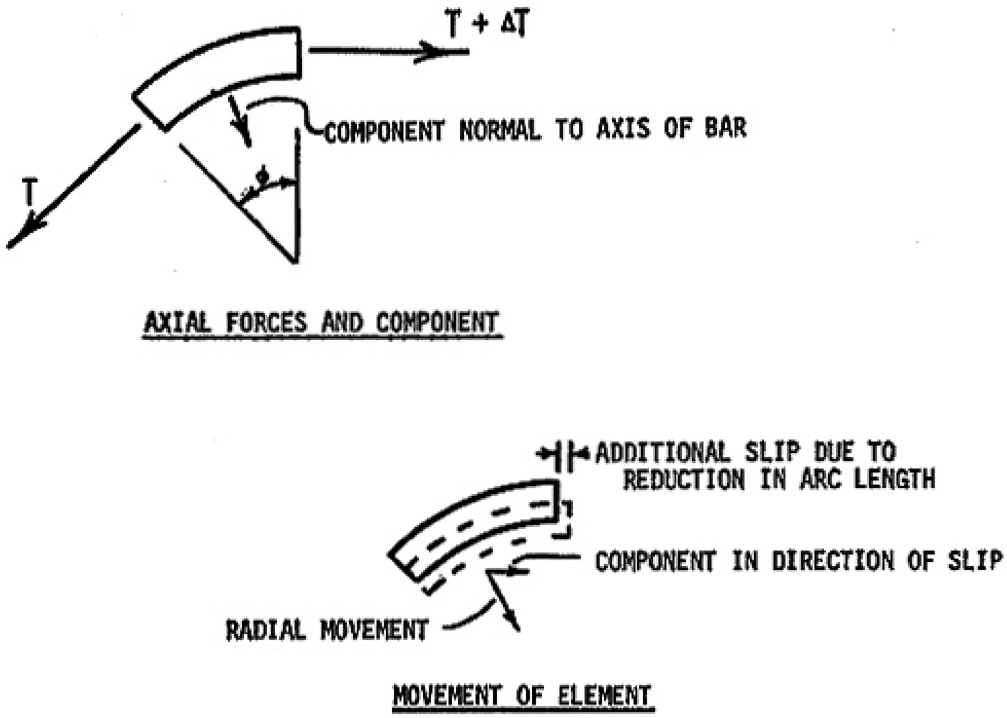


Figure 3-9: Slip due to curved section of bar (Minor and Jirsa 1971)

Minor observed that the mode of failure is affected by the angle of bend. The test results show that the bars with bends greater than 90° failed by the fracture of the concrete block more often than bent bars with smaller angles.

3.2.1.2 Radius of bend

Figure 3-10 and 3-11 show load-slip curves for #7 and #5 bars. In those cases, the variables are the radius of bend on each bar. As can be observed, the bars with the smaller radius of bend have greater slip values at a given stress.

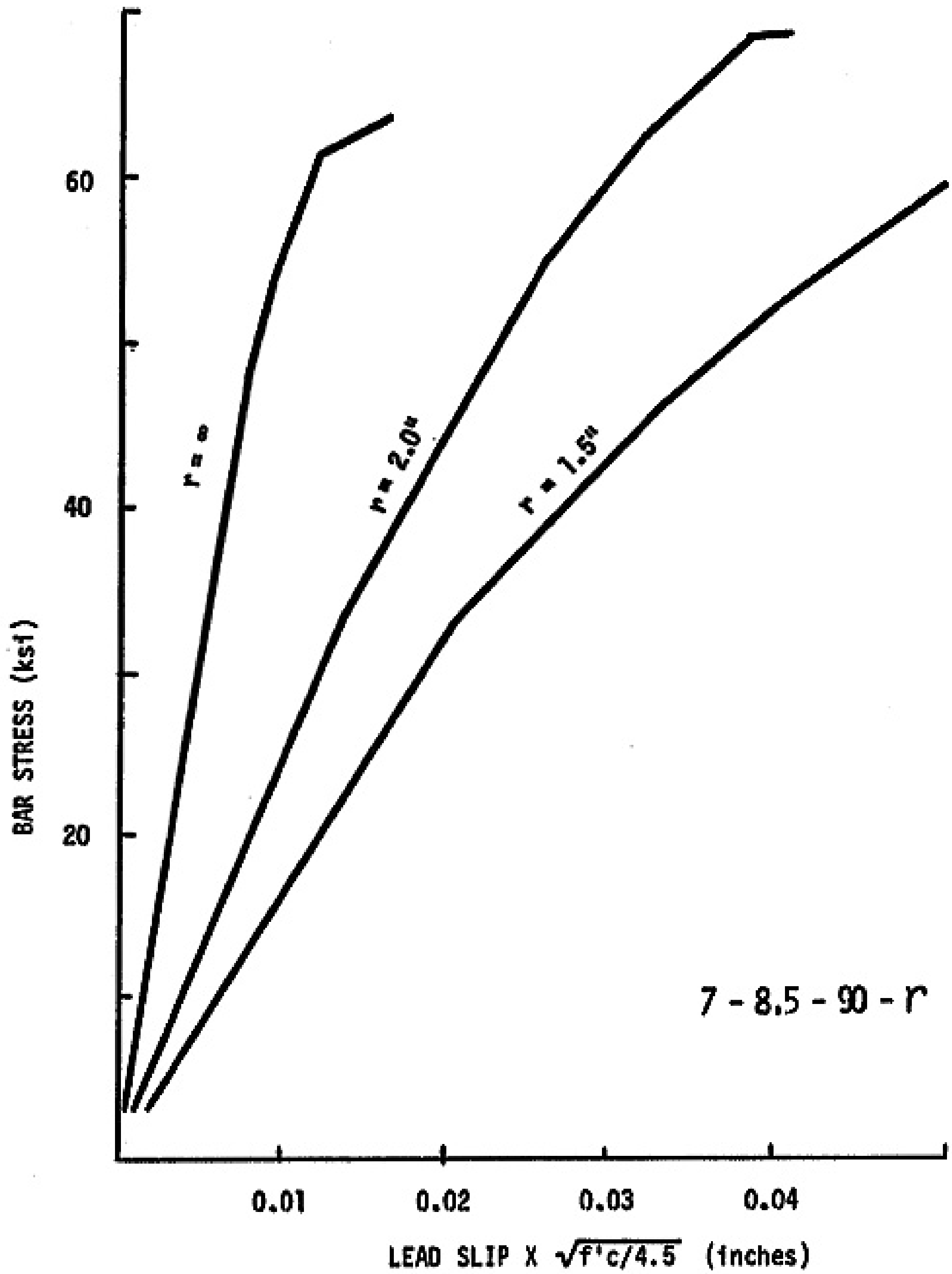


Figure 3-10: Effect of radius of bend on load-slip curves - #7 bars
(Minor and Jirsa 1971)

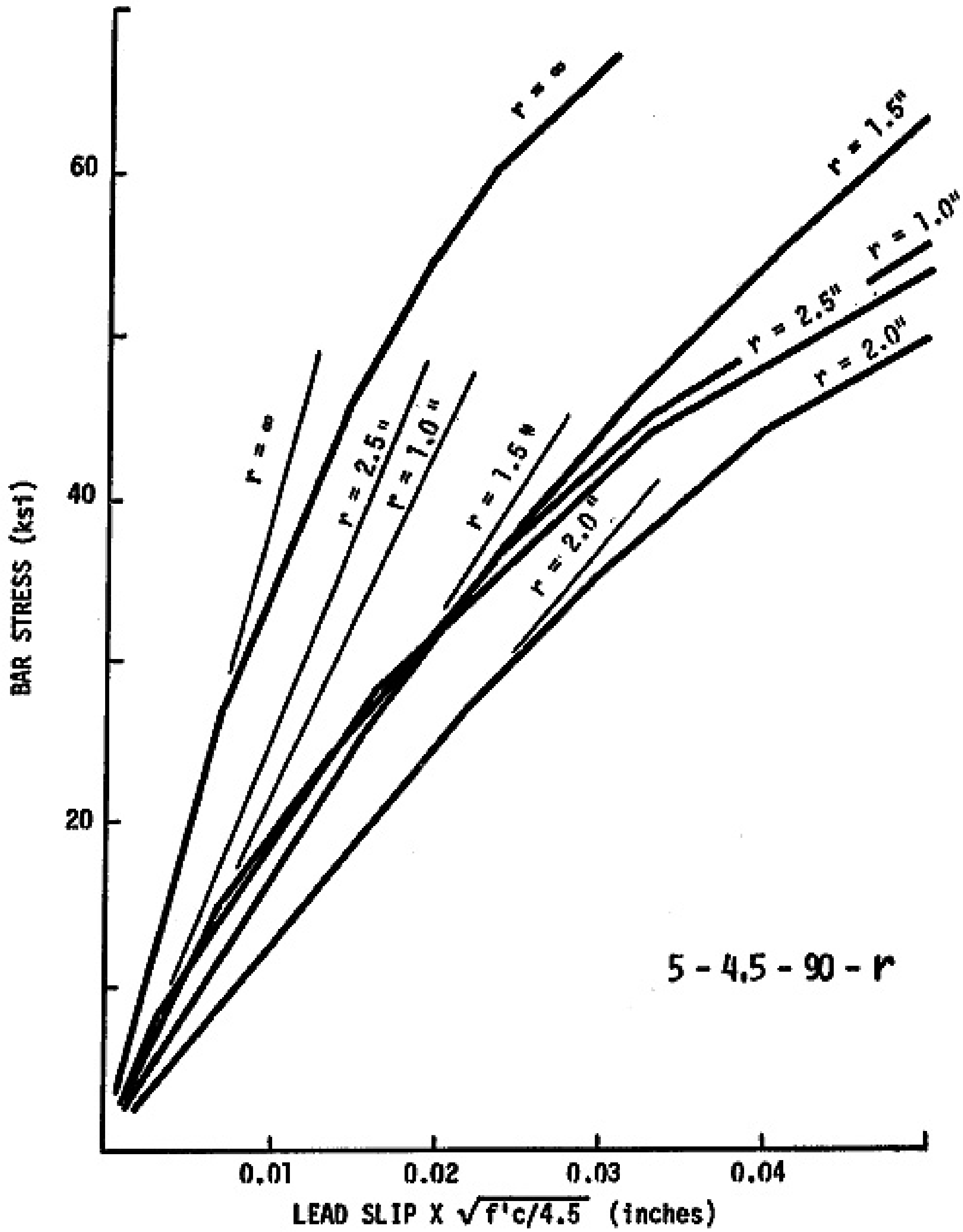


Figure 3-11: Effect of radius of bend on load-slip curves – #5 bars (Minor and Jirsa 1971)

Figure 3-12 shows the effect of radius of bend at a given strength. Although the test results show considerable scatter, the trend is that the greater slip occurs when the r/d ratios (inside radius to bar diameter) are smaller.

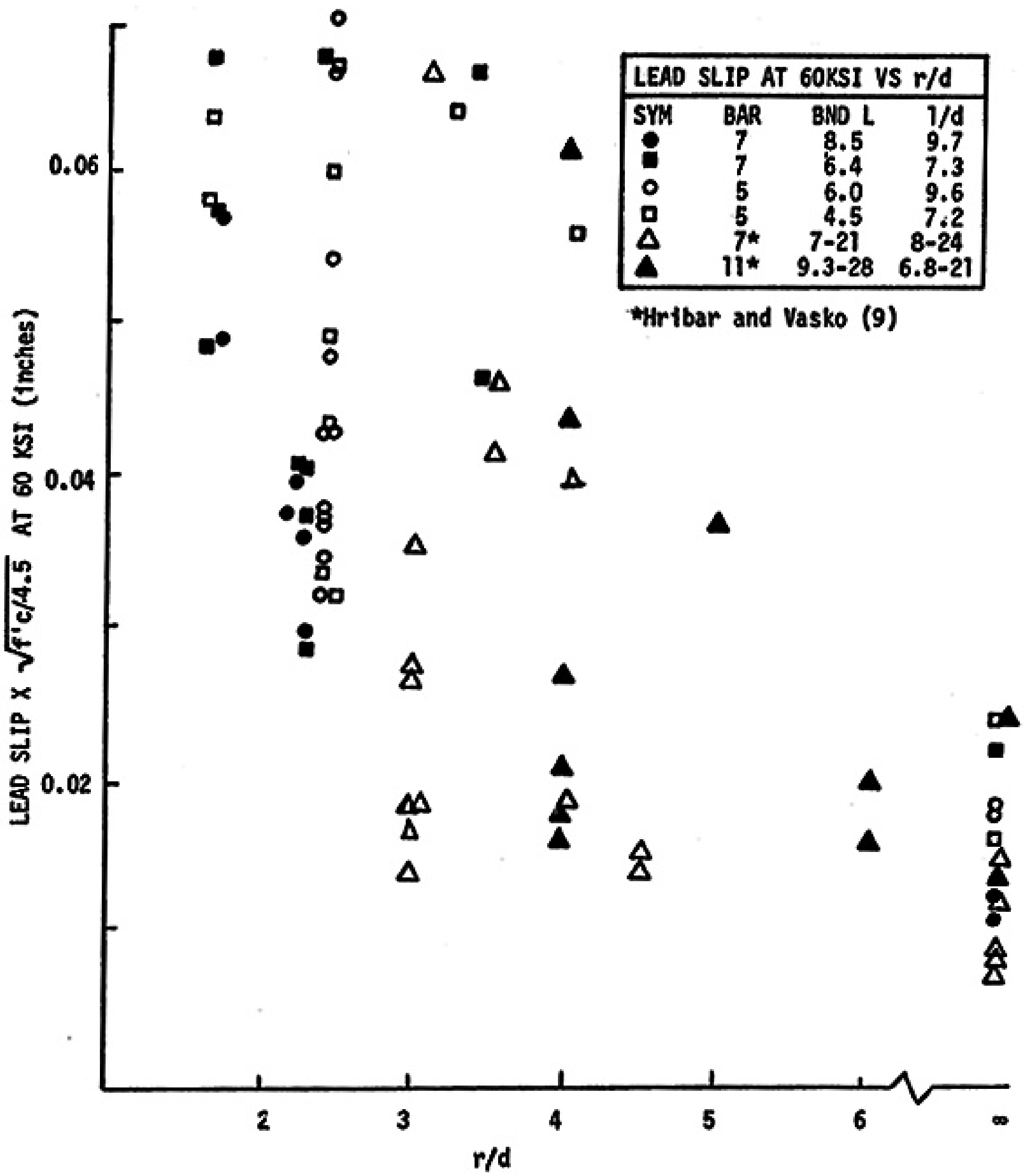


Figure 3-12: Effect of radius on slip at 60 ksi (Minor and Jirsa 1971)

3.2.1.3 Bond lengths and bar diameter

As mentioned before, the initial slope of the load slip curves were independent of the bond length. Ultimate strength for #5bars with different l/d ratios (bond length to bar diameter) are shown in Figure 3-13. As expected, the specimens with larger bond length give higher ultimate strengths.

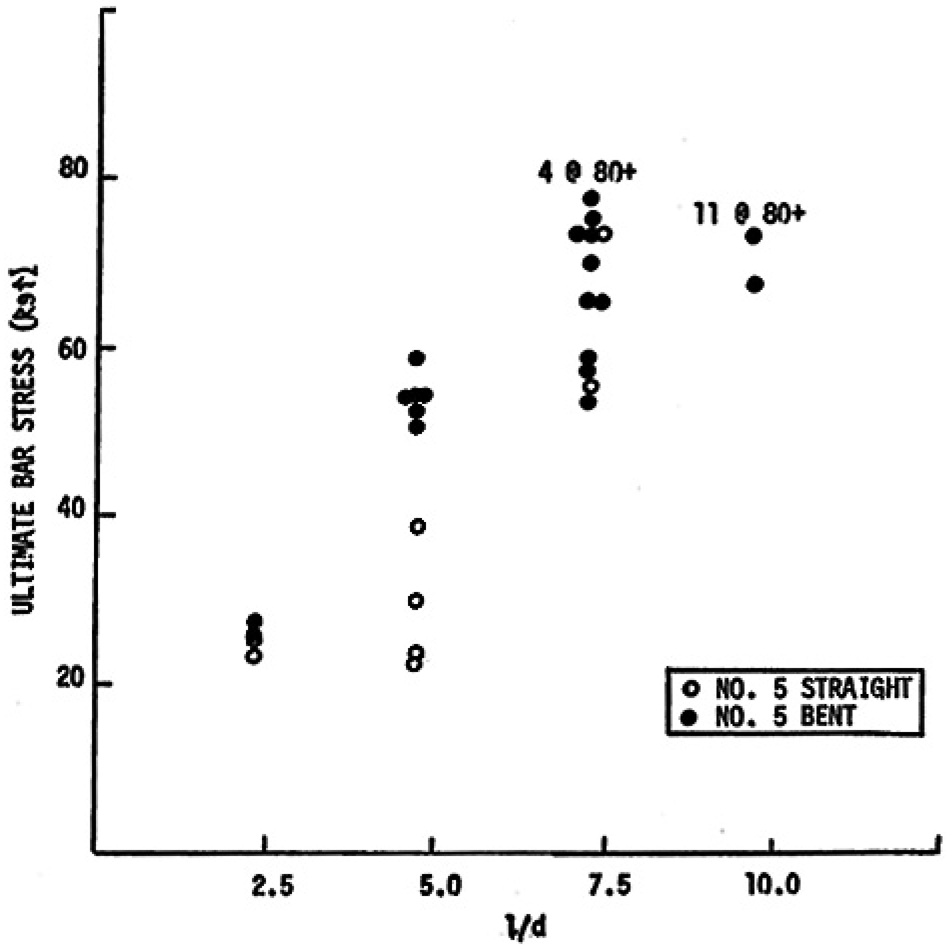


Figure 3-13: Ultimate strength - #5 bar (Minor and Jirsa 1971)

If the bar has a sufficient bond length, the fracture failure of bar occurs before pullout failure of the bar.

3.2.2 Marques and Jirsa – A study of hooked bar anchorages in beam-column joints (Marques and Jirsa 1972; Marques and Jirsa 1975)

A study was conducted to examine some of the factors influencing the anchorage capacities of hooked bars in beam-column joints of reinforced concrete structures. The study was divided into two phases. Previously, Minor evaluated short bent bars anchored in small concrete blocks. The second phase involves tests of twenty-two specimens which are full-scale models of beam-column joints in a structure and represent a more realistic condition of hooked bar anchorages. To evaluate the capacity of hooked bar anchorage in the beam, column axial load, vertical column reinforcement, side concrete cover, and lateral reinforcement through the joint were considered.

3.2.2.1 Influence of column axial load

The stress-slip curves for influence of column axial load are shown in Figure 3-14. The only variable is the level of column axial load. Based on the test result, there is no specific influence of column axial loads. In this case, the tail of the hook was oriented in the direction of the axial load. Thus, Marques indicated that other orientations of bent bars and different lateral confinement might produce different results.

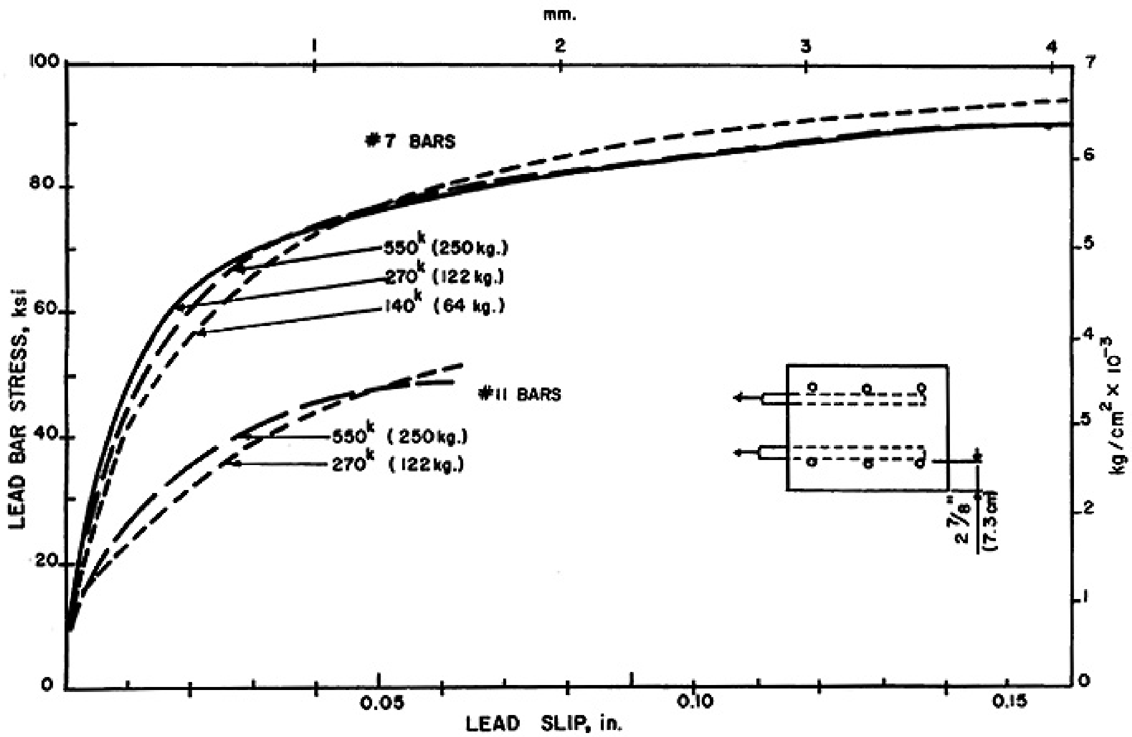


Figure 3-14: Influence of column axial load on slip (Marques and Jirsa 1975)

3.2.2.2 Influence of bend angle

Figure 3-15 shows the stress-slip curves for bend angle of bars. In this case, the variables are only bend geometry, but axial load and lateral confinement are constant. At a given stress, 90° hooked bars tend to be stiffer than 180° hooked bars.

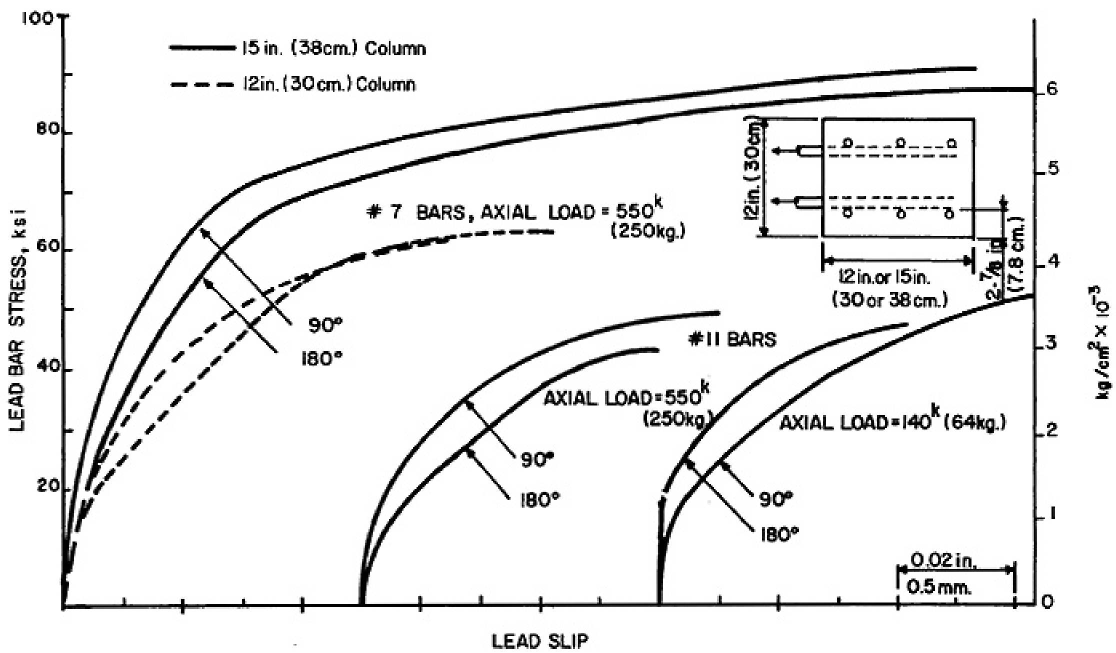


Figure 3-15: Influence of bend angle on slip (Marques and Jirsa 1975)

3.2.2.3 Influence of lead embedment

The lead embedment of bars is defined as the straight bar between the hook and the column face. The influence of the lead embedment is shown in Figure 3-16. The test results indicates that the lead slip is greater for bars with shorter lead embedment. Marques mentioned that for large bars the lead embedment in these tests provided a short length for stress transfer to the concrete ahead of the hook. However, with larger lead embedment, the lateral restraint against splitting is improved because a larger area of concrete must spall or split before the bar will fail.

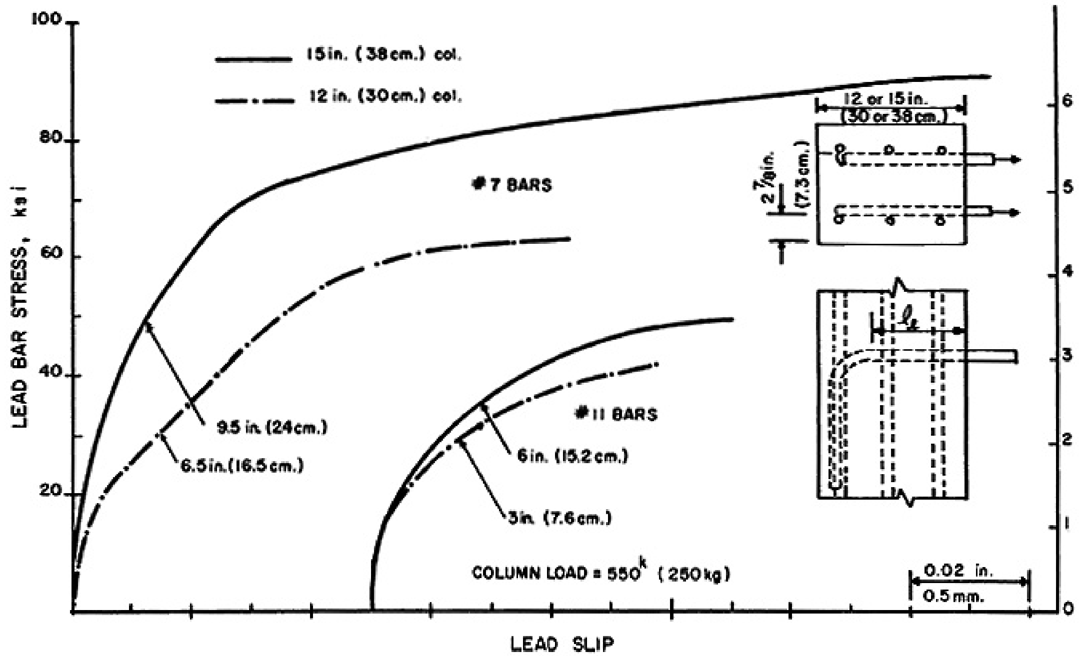


Figure 3-16: Influence of lead embedment on slip (Marques and Jirsa 1975)

3.2.2.4 Influence of lateral confinement

The influence of lateral confinement provided by the column bars, the side concrete cover, and ties through the joint was studied. Figure 3-17 shows the stress-slip curves for #7 bars with a 90° hook. The curves show almost the same stress-slip relationships and indicate that there is little difference in slip between four specimens with different column reinforcement arrangement at a given bar stress.

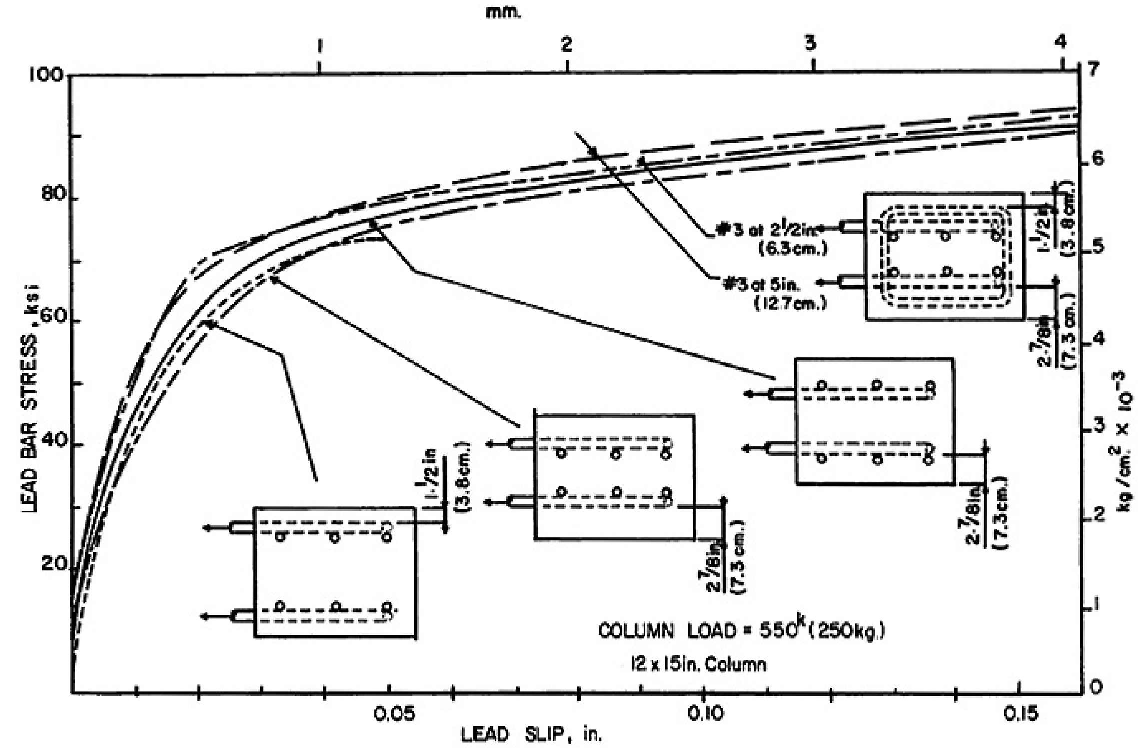


Figure 3-17: Influence of confinement on slip, #7 bars, 90° hook (Marques and Jirsa 1975)

The stress-slip curves for #11 bars are shown in Figure 3-18. The test results indicate that location of the column bars has little influence on slip at a given stress, while ties through the joints increase the strength. Thus, the ties through the joint are beneficial. For the concrete cover, a smaller concrete cover resulted in a decrease in both strength and deformation capacity of the hooked bars.

Based on these observations, Marques suggested that a combination of ties through the joint and column bars outside the anchored bars would have improved the response further, because the ties would provide support for the column bars.

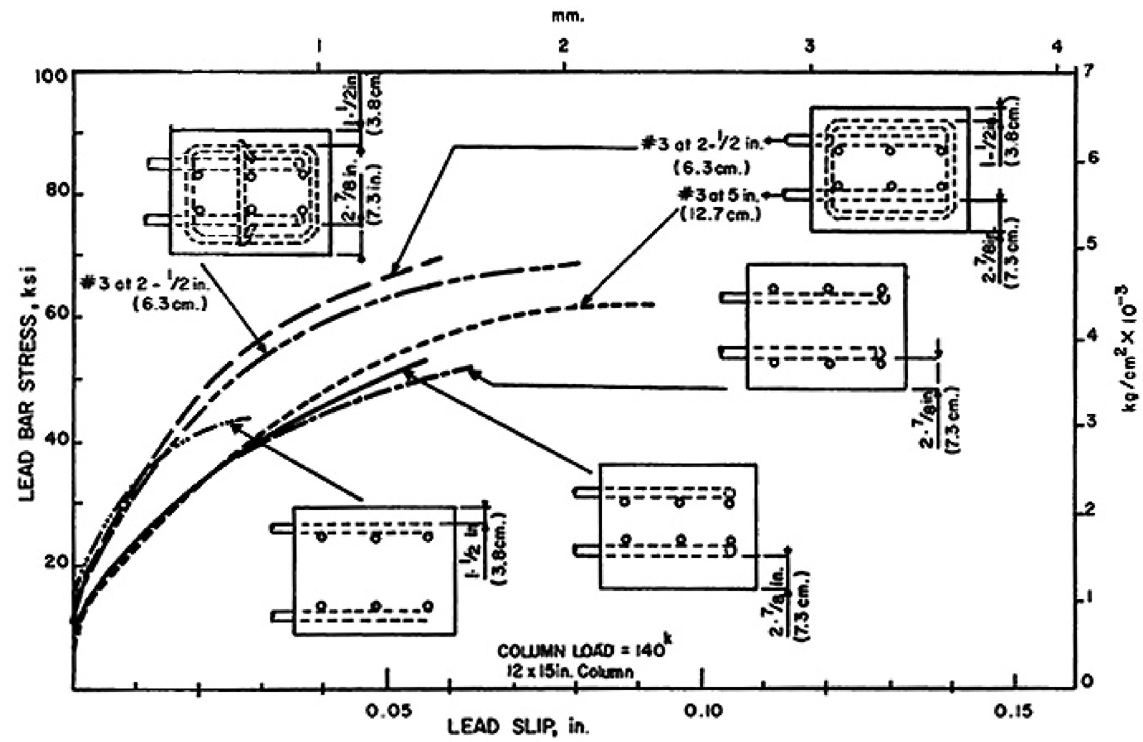


Figure 3-18: Influence of confinement on slip, #11 bars, 90° hook

(Marques and Jirsa 1975)

3.2.3 Pinc, Watkins and Jirsa – strength of hooked bar anchorages in beam-column joints (Pinc et al. 1977)

A total of sixteen specimens were tested to evaluate the influence of lead embedment and lightweight aggregate concrete on the strength of hooked bar anchorage in beam-column joints. The specimens were full scale models. The test specimens were patterned after the study by Marques and Jirsa so that the results could be compared.

3.2.3.1 Influence of lead embedment

3.2.3.1.1 Influence of lead embedment length on slip

The influence of lead embedment on slip for #9 bars and #11 bars is shown in Figures 3-19 and 3-20, respectively. The test results indicate that the strength and stiffness of the hooked bars depend on the lead embedment or by the thickness of the column. In general, the longer the lead embedment lengths, the higher the stress reached. Shorter lead embedments slip more at all stress levels.

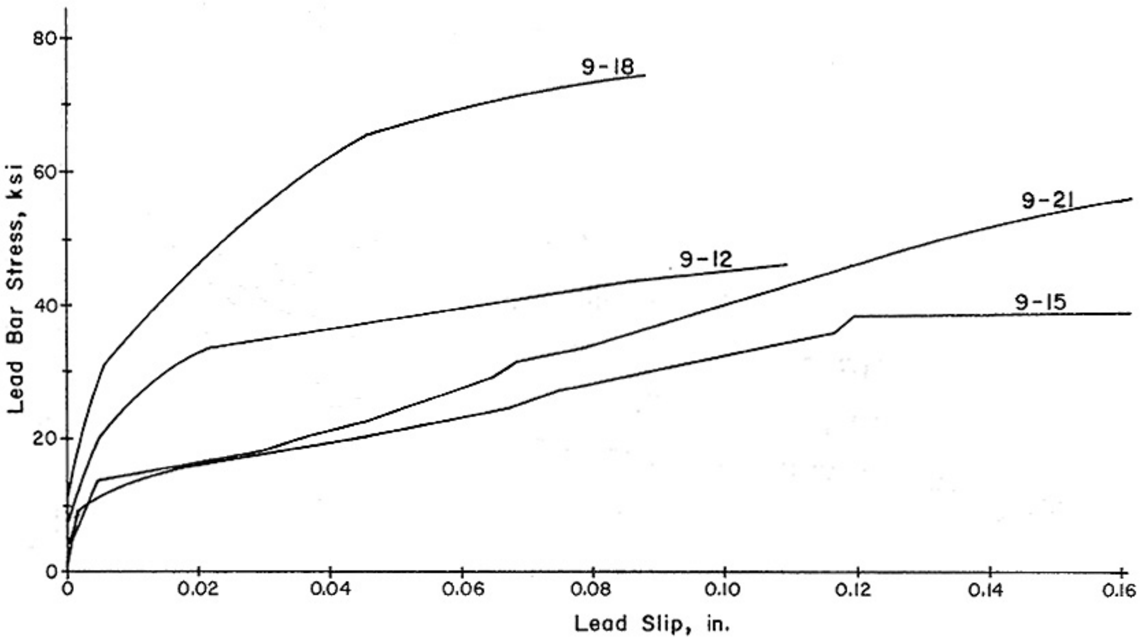


Figure 3-19: Influence of lead embedment on slip for #9 bars (Pinc et al. 1977)

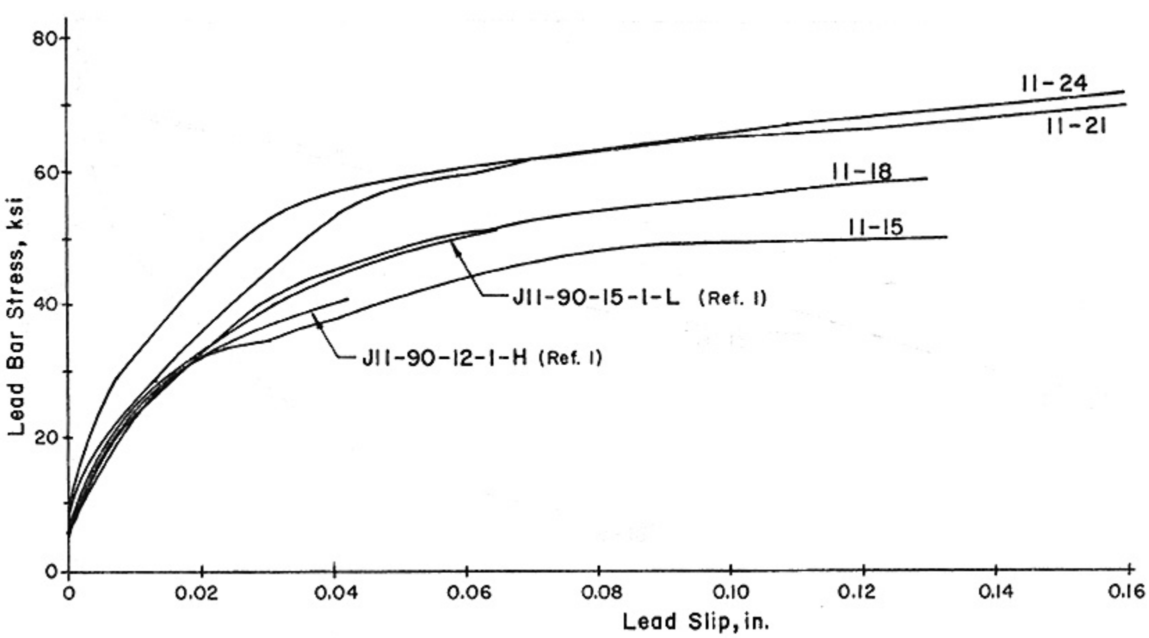


Figure 3-20: Influence of lead embedment on slip for #11 bars (Pinc et al. 1977)

3.2.3.1.2 Influence of lead embedment length on stress characteristics

Figures 3-21 and 3-22 show the stress measured at the start of the hook plotted against the measured lead bar stress for #9 bars and #11 bars. Pinc et al. note that at a given level of stress, the difference between the lead bar stress and the stress at the start of the hook may be considered as the amount of stress that is transferred to the concrete by the straight lead embedment. The test results indicate that specimens with short lead embedment transferred much less stress along the straight bar portion than specimens with longer lead embedment.

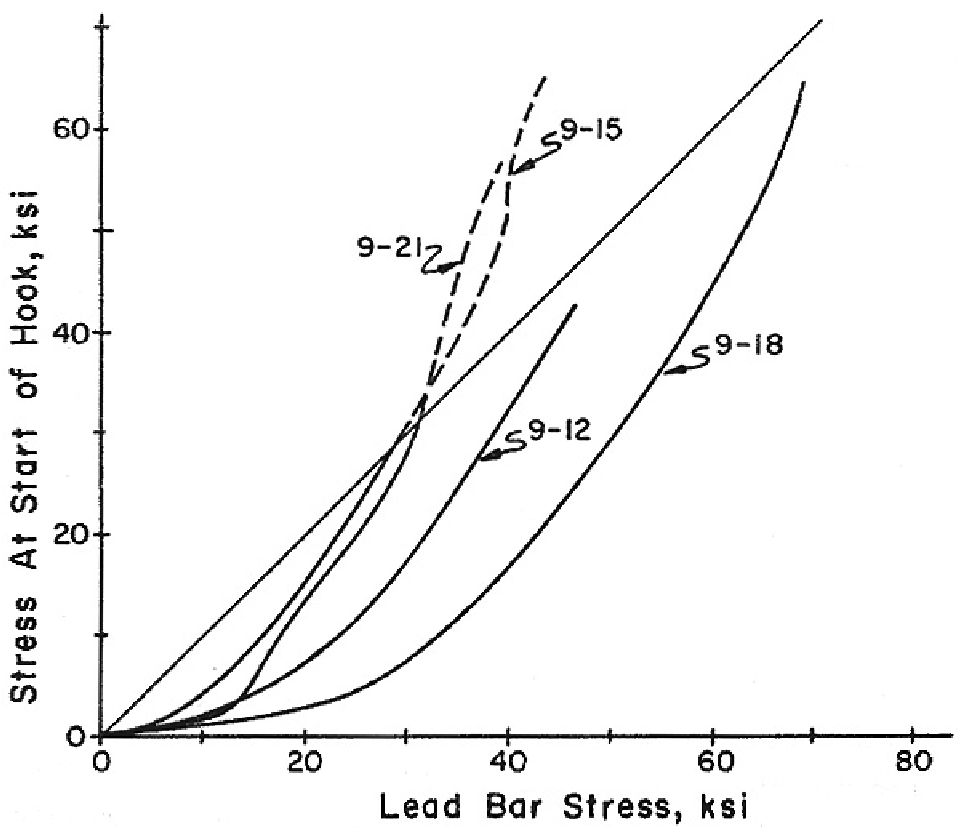


Figure 3-21: Influence of lead embedment on stresses for #9 bars (Pinc et al. 1977)

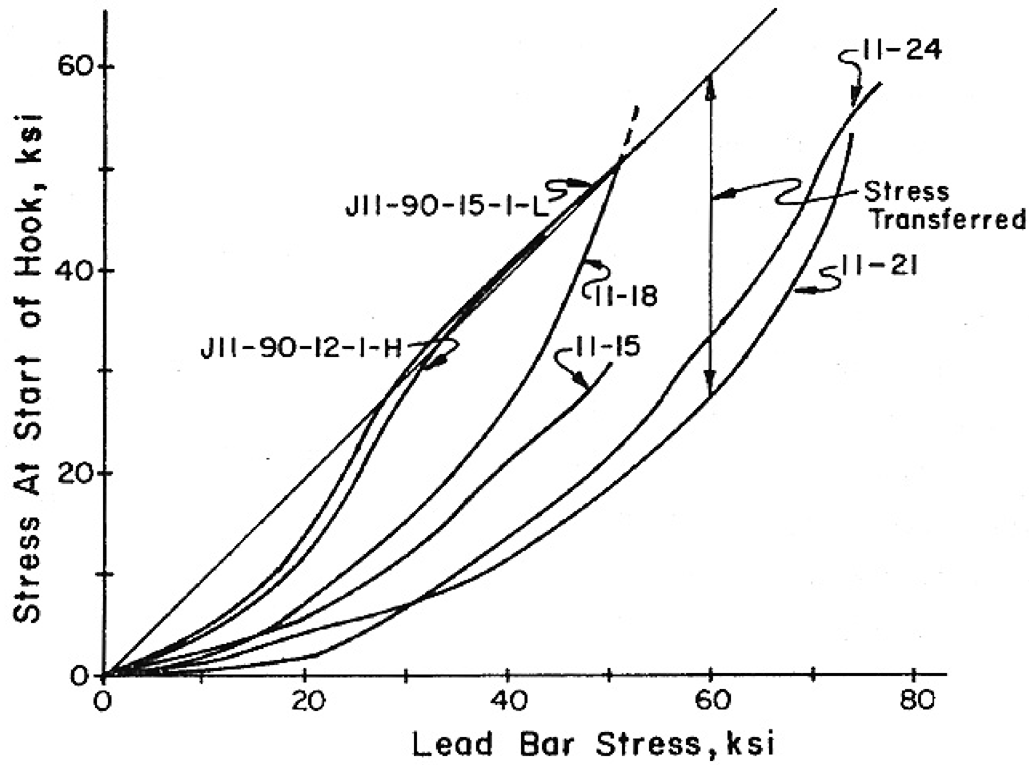


Figure 3-22: Influence of lead embedment on stresses for #11 bars (Pinc et al. 1977)

3.2.3.2 Lightweight concrete

3.2.3.2.1 Influence of hook geometry

Figure 3-23 shows the effect of hook geometry. Two specimens had 50 percent of the aggregate replaced by lightweight materials and two were normal weight concrete. The results indicate that there is little difference in the strength of 90° and 180° hooks. However, slip in lightweight concrete is significantly greater than in normal weight specimens.

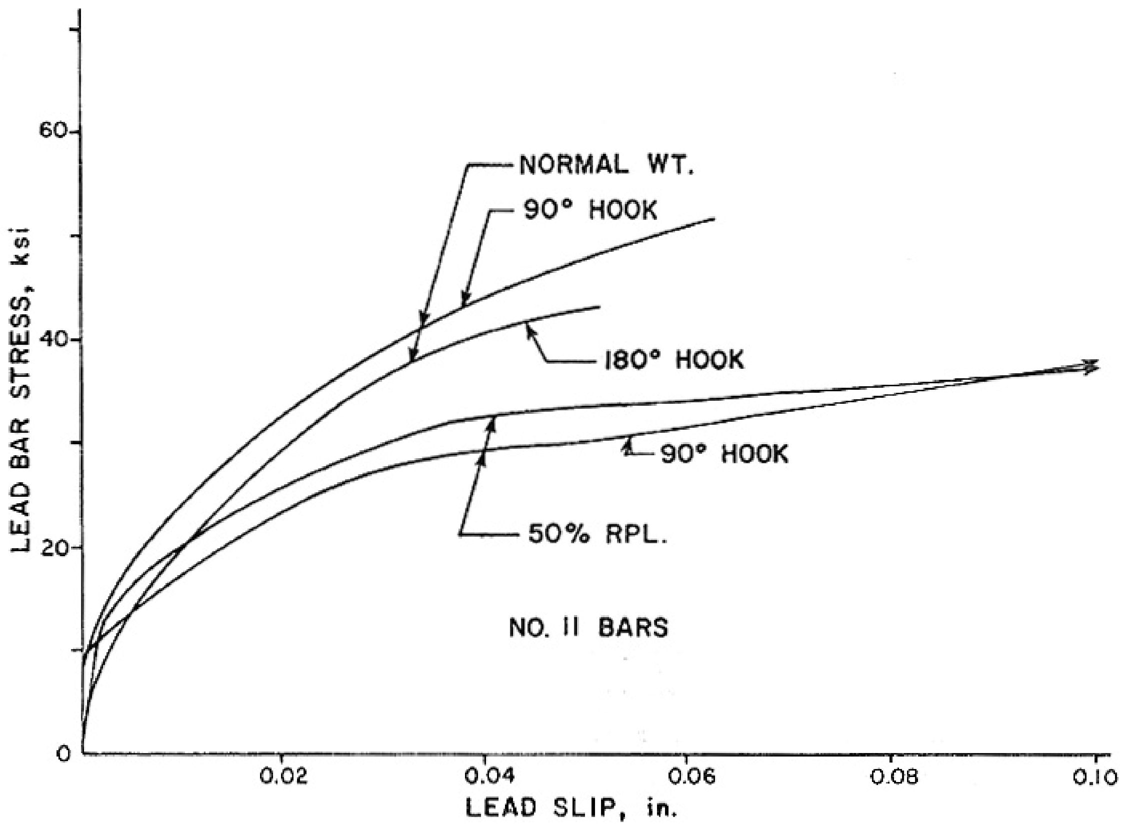


Figure 3-23: Influence of hook geometry on slip (Pinc et al. 1977)

3.2.3.2.2 Influence of confinement

The test results for the influence of Type 3 confinement, #3 closed ties at 5 in. spacing through the joint, and Type 1 confinement, no ties through the joint, are presented in Figure 3-24. The inclusion of ties through the joint on the two normal weight concrete specimens shows an increase in the stress and slip at failure. The two 50 percent replacement lightweight concretes result in similar behavior; but it require increased slips to attain higher strength.

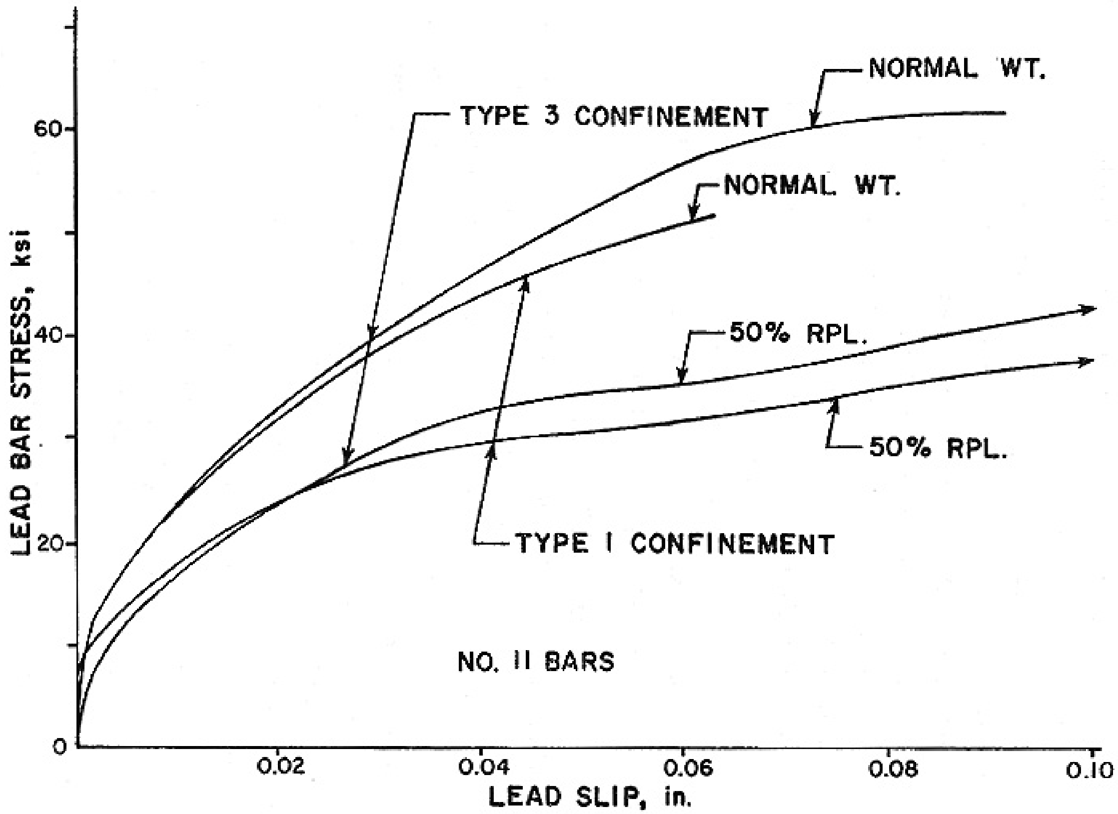


Figure 3-24: Influence of confinement on slip (Pinc et al. 1977)

3.2.3.2.3 Influence of axial load

As mentioned before (Marques and Jirsa 1972; Marques and Jirsa 1975), the effect of axial load is negligible for normal weight as well as lightweight concrete specimens. It should be noted that the tail extension of the hooked bar was oriented in the direction of application of axial load in all four cases. Other orientations of bent bars and different lateral confinements might produce different results.

3.2.3.2.4 Influence of concrete mix

To evaluate the effect of concrete mix with lightweight fine aggregates, three mixes were used: all lightweight fine aggregate, replacement of 50 percent of the lightweight fine aggregates with normal weight sand aggregate, or replacement of all of the lightweight fine aggregate with normal weight sand aggregate.

a) All-lightweight concrete

As seen in Figures 3-25 and 3-26, the strength of the all-lightweight concrete specimen decreased approximately 15% compared to the normal weight concrete.

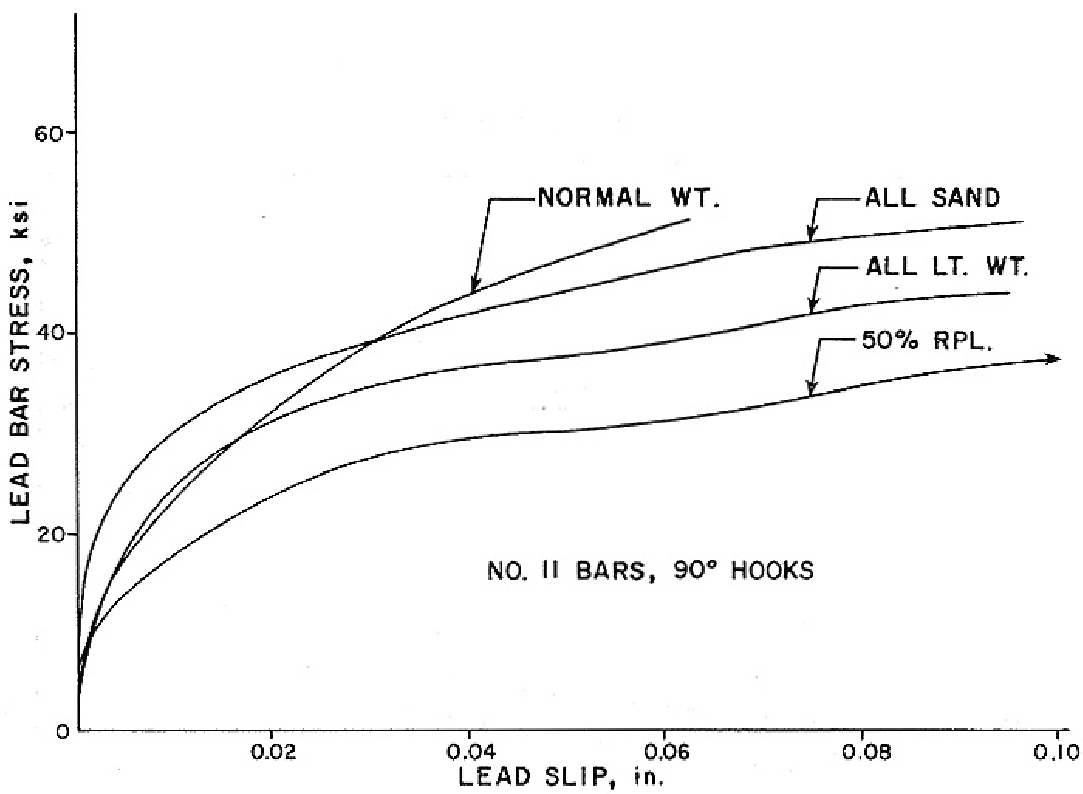


Figure 3-25: Influence of lightweight concrete mixes on slip for #11 bars (Pinc et al. 1977)

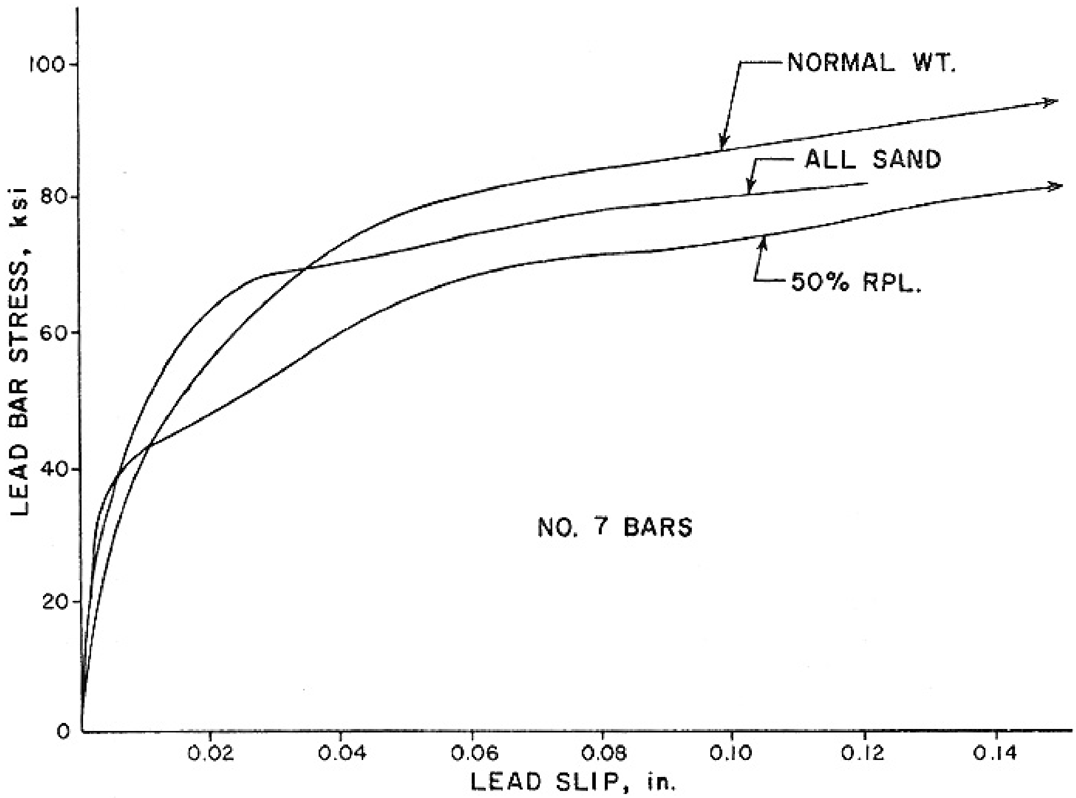


Figure 3-26: Influence of lightweight concrete mixes on slip for #7 bars (Pinc et al. 1977)

At failure the all-lightweight concrete specimen slipped more than the normal weight specimen.

b) Fifty percent replacement

For #11 bars, Figure 3-25 shows that the 50 percent replacement concrete specimen attains approximately 75 percent of the strength of the normal weight concrete specimen. Also, this specimen shows an increase in slip, approximately 80 percent, compared to the normal weight concrete specimen at failure.

As seen in Figure 3-26 for #7 bars, the 50 percent replacement concrete specimen attains approximately 82 percent of the strength of the normal weight concrete specimen. At failure, however, the slip of the 50 percent replacement concrete specimen has almost same response on the normal weight specimen.

c) All-sand lightweight concrete

In Figure 3-25, the ultimate stress for the all-sand lightweight concrete specimen reaches roughly the same ultimate stress level of the normal weight concrete specimen. At failure, the slip on the all-sand lightweight concrete specimen was 50 percent greater than the normal weight concrete specimen. The behavior of the all-sand lightweight concrete specimen is presented in Figure 3-26. The curve indicates that the all-sand lightweight concrete specimen behaves very similar to the normal weight concrete specimen.

3.2.4 Hamad – Effect of epoxy coating on bond and anchorage of reinforcement in concrete structures (Hamad 1990; Hamad et al. 1993)

Twenty-five specimens were tested to evaluate the behavior and anchorage capacity of epoxy-coated hooked bars in beam-column joints. To investigate the effects of epoxy-coated bars relative to uncoated bars, variables included bar size, concrete strength, concrete cover, lateral reinforcement through the joint, and hook geometry on the hooked bar anchorage.

3.2.4.1 Effect of bar size

As shown in Figure 3-27, the effect of bar size is similar for the anchorage behavior of epoxy-coated hooked bars compared to uncoated bars.

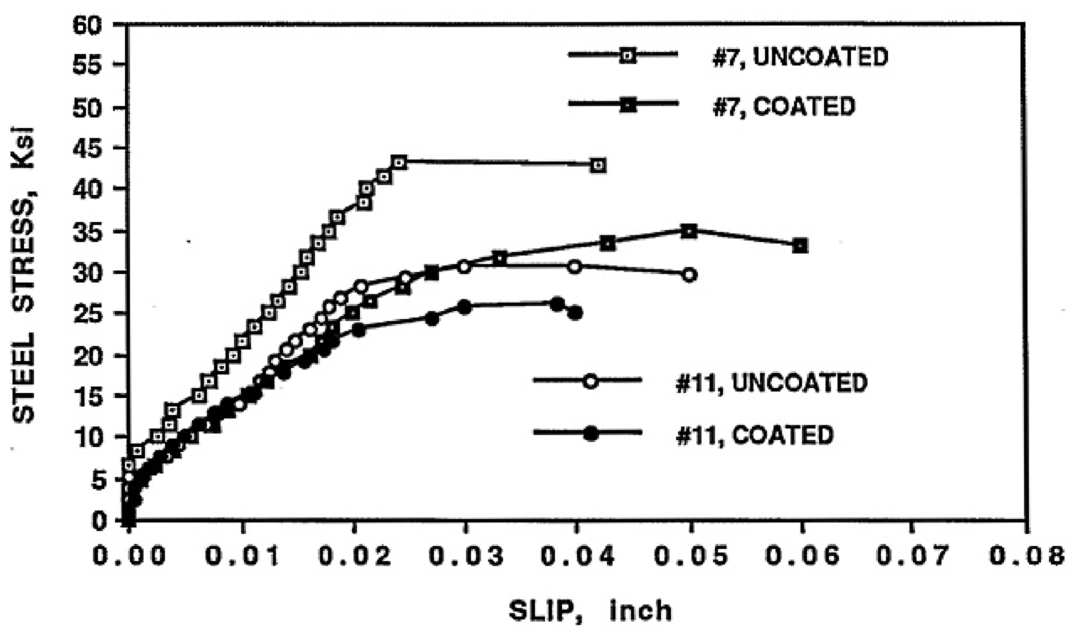
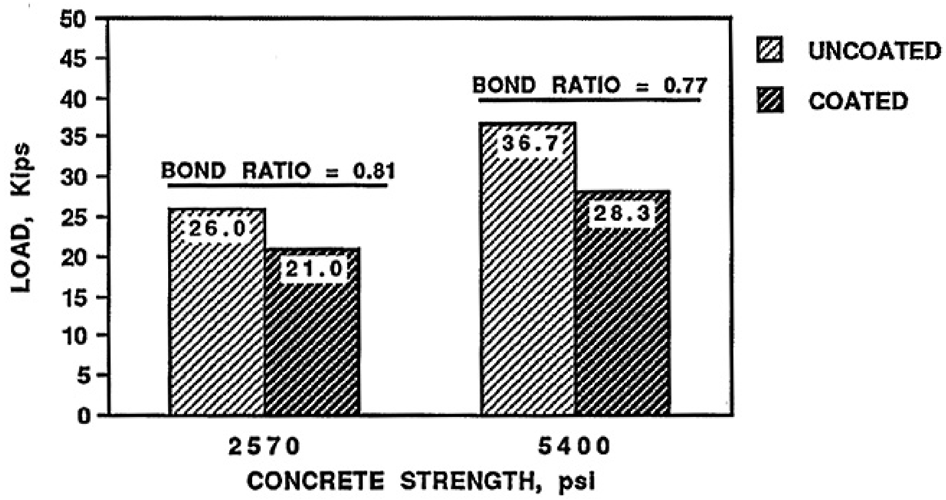


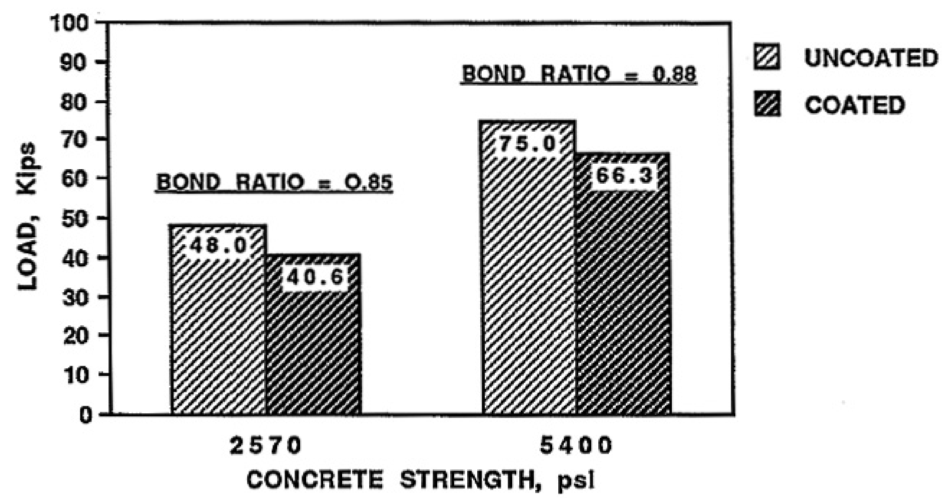
Figure 3-27: Effect of bar size on steel stress-slip behavior of uncoated and epoxy-coated 90° hooked bar, f'_c = 2570 psi (Hamad 1990)

3.2.4.2 Effect of concrete strength

The strength of #7 bars or #11, uncoated or epoxy-coated bars increased as the concrete strength increased. The results are shown in Figure 3-28. The epoxy-coating decreased in the anchorage strength of the hooked bars by less than 25%.



(a) No.7 hooked bars



(b) No.11 hooked bars

Figure 3-28: Effect of concrete strength on anchorage capacities of uncoated and epoxy-coated 90° hooked bars (Hamad 1990)

3.2.4.3 Effect of concrete cover

Figure 3-29 shows the effect of concrete cover for a #7 hooked bar, uncoated or epoxy-coated.

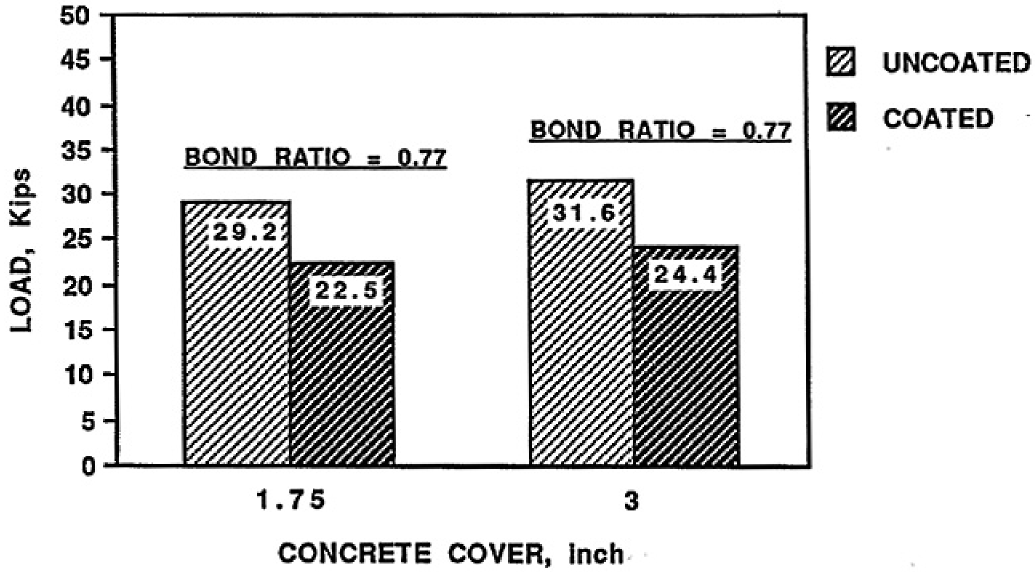


Figure 3-29: Effect of concrete cover on anchorage capacities of #7 uncoated and epoxy-coated 90° hooked bars, loads are normalized at f'_c = 4000 psi (Hamad 1990)

Hamad indicated that: *the reduced concrete cover caused a reduction in the lateral confinement of the joint region and its restraint against splitting. However, the variation of the level of confinement by concrete cover did not affect the amount of reduction of anchorage strength of epoxy-coated bars.*

3.2.4.4 Effect of joint ties

Previous studies verified that the ties though the joint region improve the load-slip behavior of bars. As shown in Figures 3-30 and 3-31, the presence of ties improved both the strength and deformation at failure of #7 bars and #11, uncoated or epoxy-coated. In addition, slip at failure of both uncoated and coated bars was more than twice the slip relative to no ties in the joint region.

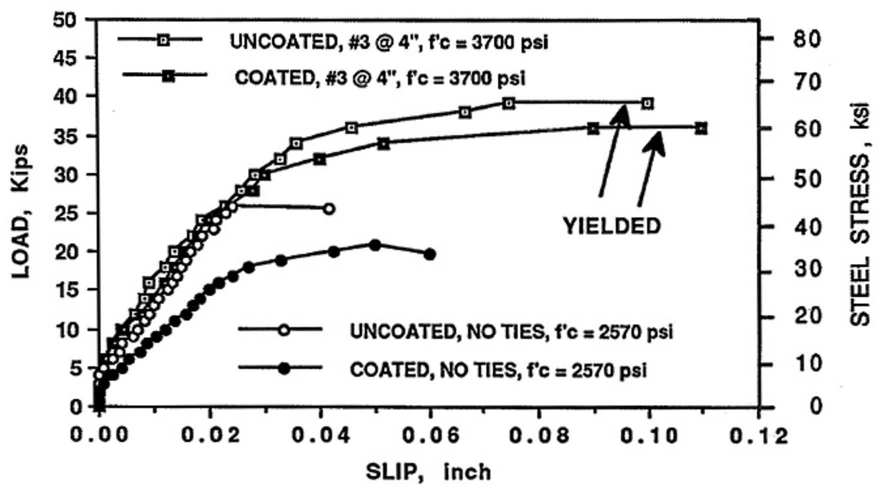


Figure 3-30: Effect of lateral reinforcement through the joint region on load-slip behavior of #7 uncoated and epoxy-coated 90° hooked bars (Hamad 1990)

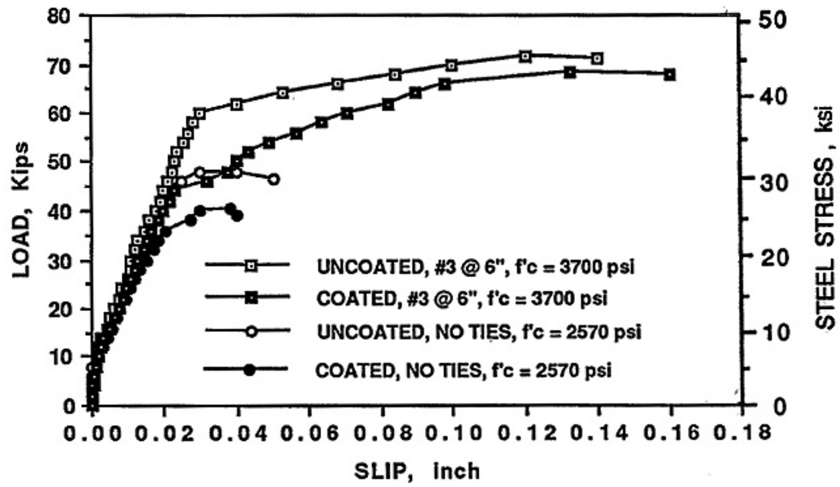


Figure 3-31: Effect of lateral reinforcement through the joint region on load-slip behavior of #11 uncoated and epoxy-coated 90° hooked bars (Hamad 1990)

3.2.4.5 Effect of hook geometry

The test result shows that there is no significant effect of hook geometry on uncoated or epoxy-coated bars.

As seen in Figures 3-32 and 3-33, for both 90° and 180° hooked bars, epoxy-coated bars develop lower capacities and more slip than uncoated bars at the same level of load. Also, for the epoxy-coated bars, the 90° hooked bars were stiffer than the 180° hooked bars at the same stress level.

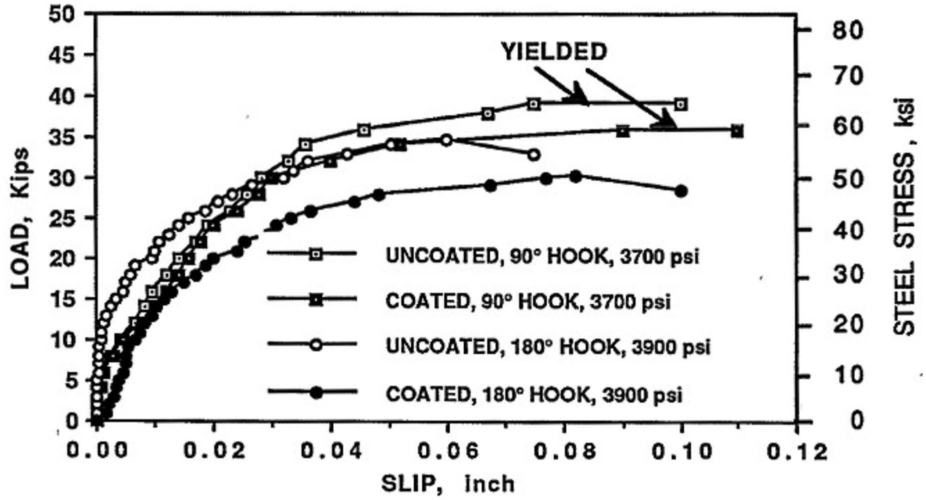


Figure 3-32: Effect of hook geometry on load-slip behavior of #7 uncoated and epoxy-coated 90° hooked bars with #3 ties at 4 in. in the joint region (Hamad 1990)

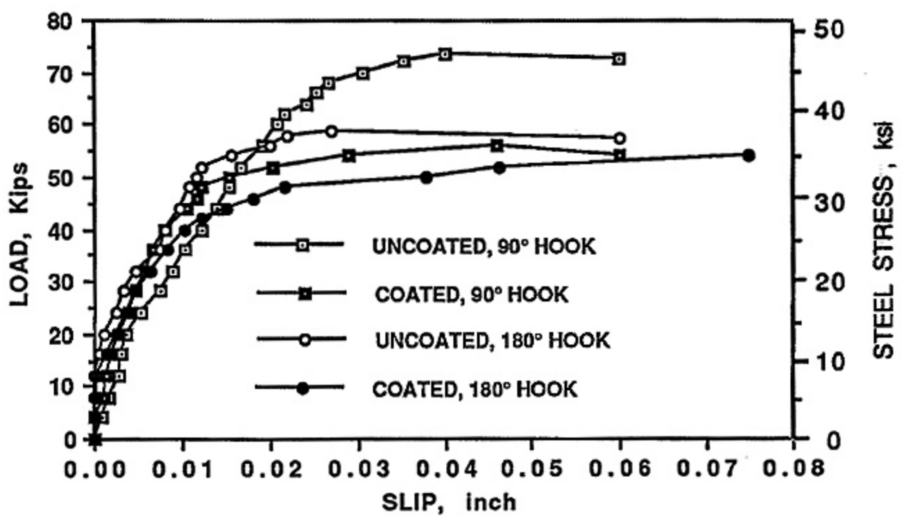


Figure 3-33: Effect of hook geometry on load-slip behavior of #11 uncoated and epoxy-coated bars, f'_c = 7200 psi (Hamad 1990)

CHAPTER 4

Design Examples of Hooked Bar Anchorage

4.1 INTRODUCTION

To illustrate the application of code provisions, development length of standard hooks in a typical RC structure and anchorages of hooked bar in a steel reinforced concrete (SRC) structure will be evaluated.

To compute the development length of a standard hook, the provisions of ACI 318-08 Section 12.5.2 are used. The basic length is multiplied by the modification factors in Section 12.5.3. The calculated length will be checked against the available dimension within the joint.

In the reinforced concrete beam-SRC column joint, the development length of hooked bar must be shortened due to the steel section placed in the column. To anchor the reinforcement in such a joint, four options will be discussed; adding more reinforcement, welding bars to a plate between flanges of steel, confinement by the shape of steel, and bearing against a plate between flanges of steel.

4.2 STANDARD HOOK FOR TYPICAL REINFORCED CONCRETE STRUCTURE

A typical exterior beam-column joint is shown in Figure 4-1. The assumed column cross-section is 24 × 24 in. and the beam is 20 × 24 in. The size of the column and beam were chosen so that the example would be a practical situation for an exterior beam-column joint.

4.2.1 Assumed structural properties

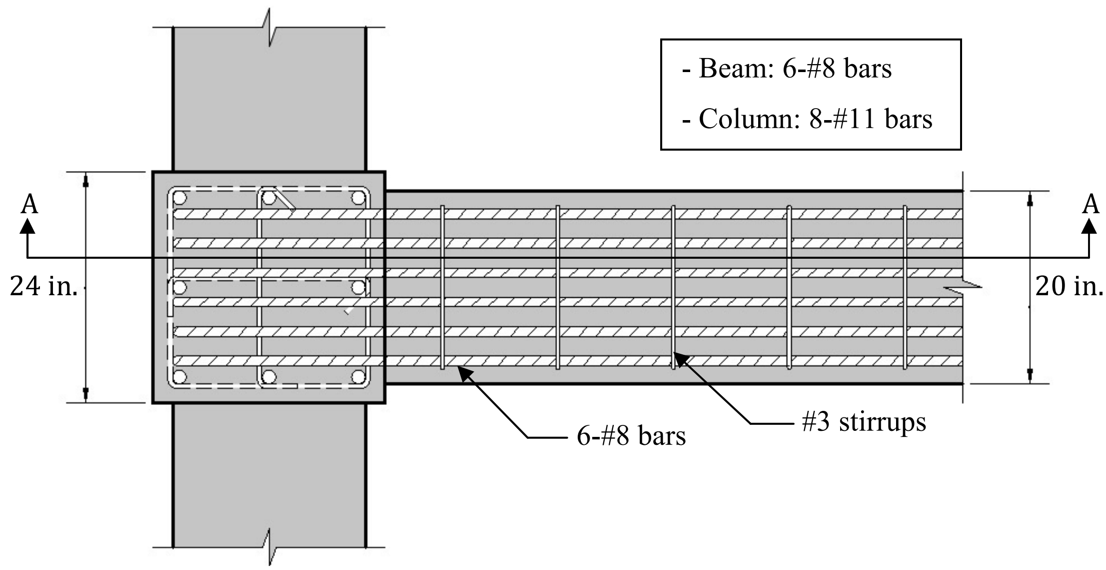
- Specified yield strength of reinforcement: $\mathrm{f_y} = 60$ ksi
- Specified compressive strength of concrete: $\mathrm{f'_c} = 4$ ksi

  (normalweight concrete)
- Reinforcement in beam: 6-#8 bars

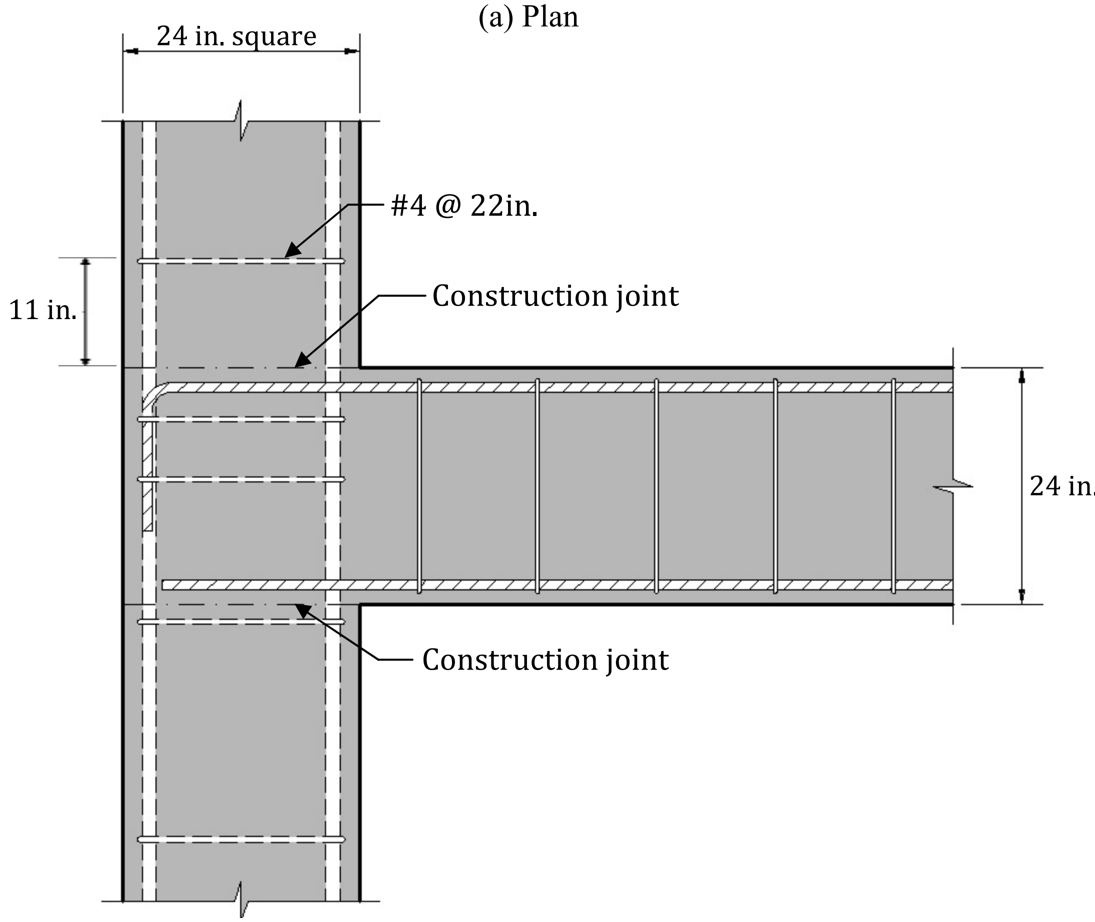
  $(\mathrm{d_b} = 1.0\ \mathrm{in.}, \mathrm{A_s} = 6 \times 0.79\ \mathrm{in.}^2 = 4.74\ \mathrm{in.}^2)$
- Ties in beam: #3 bars $(\mathrm{d_b} = 0.375\ \mathrm{in.}, \mathrm{A_v} = 2 \times 0.11\ \mathrm{in.}^2 = 0.22\ \mathrm{in.}^2)$
- Reinforcement in column: 8-#11 bars (ρ_g= 0.022)
- Hoop in column: 3-#4 bars@ 22 in.

  $(\mathrm{d_b} = 0.5\ \mathrm{in.}, \mathrm{A_v} = 3 \times 0.2\ \mathrm{in.}^2 = 0.6\ \mathrm{in.}^2)$
- Hoop in joint: 2-#4 bars@ 22 in.

  $(\mathrm{d_b} = 0.5\ \mathrm{in.}, \mathrm{A_v} = 2 \times 0.2\ \mathrm{in.}^2 = 0.4\ \mathrm{in.}^2)$
- Clear cover for column and beam: 1.5 in.



(a) Plan



(b) A-A section

Figure 4-1: Exterior beam-column joint

4.2.2 Anchorage for top reinforcement in beam

1. Compute the development of straight deformed bars in tension

A calculation will be made to see if it is possible to use straight bar anchorage in the joint.

The equation for development length is shown in Table 12.2.2 and Section 12.2.3 of ACI 318-08 here. The development length, l_d, is determined by one of these equation and l_d shall not be less than 12 in.

Table 4-1: The development length (ACI 318-08 Table 12.2.2)

Spacing and cover	No. 6 and smaller bars and deformed wires	No. 7 and larger bars
Clear spacing of bars or wires being developed or spliced not less than d_b, and stirrups or ties throughout l_d not less than the Code minimum or Clear spacing of bars or wires being developed or spliced not less than $2d_b$ and clear cover not less than d_b	$\left(\frac{f_y \psi_t \psi_e}{25\lambda\sqrt{f'_c}}\right) d_b$	$\left(\frac{f_y \psi_t \psi_e}{20\lambda\sqrt{f'_c}}\right) d_b$
Other cases	$\left(\frac{3f_y \psi_t \psi_e}{50\lambda\sqrt{f'_c}}\right) d_b$	$\left(\frac{3f_y \psi_t \psi_e}{40\lambda\sqrt{f'_c}}\right) d_b$

In ACI 318-08 Section 12.2.3,

$$l_d = \left(\frac{3 f_y \psi_t \psi_e \psi_s}{40 \lambda \sqrt{f'_c} \left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b \qquad \text{(Eq. 4-1)}$$

$$\frac{c_b + K_{tr}}{d_b} \leq 2.5 \qquad \text{(Eq. 4-2)}$$

$$\mathrm{K_{tr}} = \frac{40 \mathrm{A_{tr}}}{\mathrm{sn}} \qquad \text{(Eq. 4-3)}$$

The factors in the equation are as follows (Section 12.2.4):

ψ_t = factor used to modify development length based on reinforcement location

(ψ_t = 1.3 for more than 12 in. of fresh concrete is cast below the development length or splice, ψ_e= 1.3 for other situation)

ψ_e = factor used to modify development length based on reinforcement coating

(ψ_e= 1.0 for uncoated bars, ψ_e= 1.2 for epoxy-coated bars)

However, the product $\psi_t\psi_e$is not greater than 1.7.

ψ_s = factor used to modify development length based on reinforcement size

(ψ_s= 0.8 for No.6 and smaller bars and deformed wires, ψ_s= 1.0 for No.7 and larger bars)

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normalweight concrete of the same compressive strength

(λ = 1.0 for normalweight concrete, λ = 0.75 for lightweight concrete)

c_b = smallest of

(a) the distance from the center of a bar or wire to nearest concrete surface

= 2.0 +1.5 + 0.375 + 0.5 = 4.375 in., or

(b) one half the center-to-center spacing of the bars or wires

$$= \frac{1}{2}\left(\frac{20 - 2 \times 2.375}{5}\right) = 1.53 \text{ in.}$$

A_{tr}= total cross-sectional area of all transverse reinforcement within spacing s that crosses the potential plane of splitting through the reinforcement being developed

= 8 × 1.56 = 12.48 in^2

s = spacing of transverse reinforcement

= (24 − 2(1.5 + 0.5 + 1.41/2)) / 2 = 9.30 in.

n = the number of bars being spliced or developed along the plane of splitting

= 6

$$\mathrm{K_{tr}} = \frac{40\ \times 12.48}{9.30 \times 6} = 8.95 \text{ in.}$$

$$\frac{c_b + K_{tr}}{d_b} = \frac{1.53 + 8.95}{1.0} = 10.48 \text{ but not more than } 2.5$$

From the above factors,

$$l_d = \left(\frac{3 f_y \psi_t \psi_e \psi_s}{40\lambda\sqrt{f'_c}\left(\frac{c_b + K_{tr}}{d_b}\right)}\right) d_b = \left(\frac{3 \times 60000 \times 1.3 \times 1.0 \times 1.0}{40 \times 1.0 \times \sqrt{4000} \times 2.5}\right) \times 1.0 = 37 \text{ in.}$$

Thus, the development length is 37 in. for the top reinforcement in beam. Since this length is greater than the width of the column, 24 in., a hooked bar anchorage is needed for shorter development length.

2. Compute the development length for standard hook in tension

The development length for standard hooks in tension is presented in ACI 318-08 Section 12.5. The factors in the equation were shown in the preceding chapter (Section 3.1).

$$l_{dh} = \frac{0.02\psi_e f_y}{\lambda\sqrt{f'_c}} d_b \geq \max\left(8\mathrm{d_{b,}}\ 6\ \mathrm{in.}\right) \qquad \text{(Eq. 4-4)}$$

From the properties of member, the development length for standard hook is

$$l_{dh} = \frac{0.02\psi_e f_y}{\lambda\sqrt{f'_c}} d_b = \frac{0.02\ \times 1.0 \times 60000}{1.0\ \times \sqrt{4000}} \times 1.0 = 19\ \text{in.}$$

Based on ACI 318-08 Section 12.5.3, the applicable factors will be applied.

(a) For #11 and smaller hooks with side cover ≥ 2.5", and 90° hook with cover on bar extension beyond hook ≥ 2.5":

- Hooks with side cover = 2 in. + 1.5 in. + 0.375 in. = 3.875 in. ≥ 2.5 in.

If the extension of hooked bar is placed near the ties in column,

- 90° hook with cover on bar extension beyond hook

  = 1.5 in. – 0.5 in. = 2.0 in. ≤ 2.5 in.

Thus, ACI Section 12.5.3.2 (a) does not apply, and the multiplier is 1.0.

(b) For 90° hooks of #11 and smaller bars enclosed within ties or stirrups parallel to the bar being developed, spaced $\leq 3d_b$ along the length of tail extension of the hook plus bend:

For ties for column, #4 ties are selected (ACI 318-08 Section 7.10.5.1). The required spacing of #4 closed ties as the following (ACI 318-08 Section 7.10.5.2):

s_{min} = smallest of 16 longitudinal bar diameters = 16 × 1.41 = 22.56 in.,

48 tie bar diameters = 48 × 0.5 = 24 in.,

least dimension of the compression member = 24 in.

Thus, vertical spacing of tie for column is 22 in. This tie spacing at joint area is greater than $3d_b$ (3 in.). Therefore, ACI Section 12.5.3.2 (b) does not apply, and so the multiplier is 1.0. Thus, the development length for hooked bar is

$$l_{dh} = 19 \times 1.0 \times 1.0 = 19 \text{ in.} \geq 8 \text{ in.}$$

The available length for hooked bar anchorage is 24 in. – 1.5 in. – 0.5 in. = 22 in. In addition, the tail of hook should be placed within the depth of the beam due to construction joint at the beam soffit and the top of the floor. The total length of tail is 4 in. + 12in. = 16 in. which is less than the depth of beam, 24 in.

The detail of hooked bar anchorage is shown in Figure 4-2.

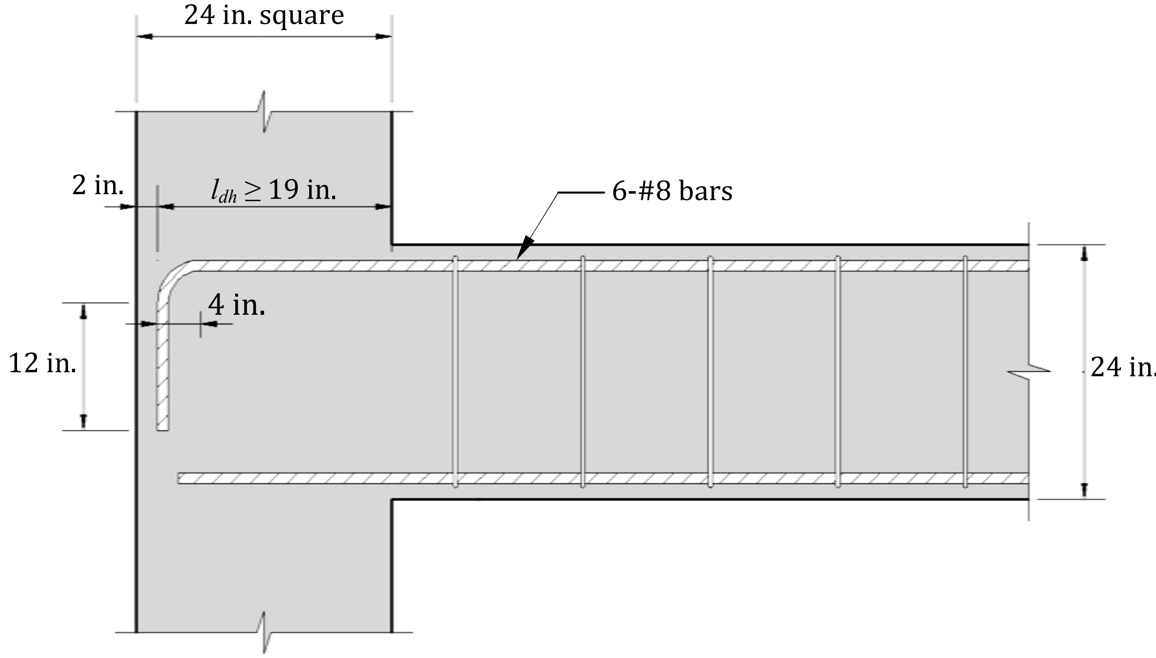


Figure 4-2: Hooked bar detail in exterior beam-column joint

4.3 HOOKED BAR IN RC BEAM-SRC COLUMN JOINT

A steel reinforced-concrete (SRC) column consists of a steel structural shape encased in a conventionally reinforced concrete column a shown in Figure 4-3 (Morino 1998; Shanmugam and Lakshmi 2001).

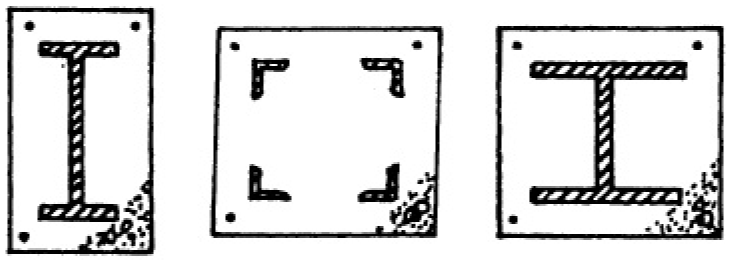


Figure 4-3: Encased steels in concrete (Morino 1998)

Because of a steel shape in the SRC column, the bars of concrete beams cannot continue through the column. Figure 4-4 shows a typical section of an SRC column. The transverse steel bars in the SRC column provide lateral restraint to the column longitudinal bars and confine the concrete.

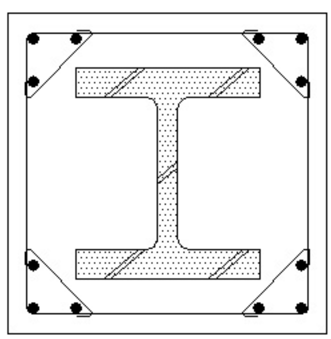
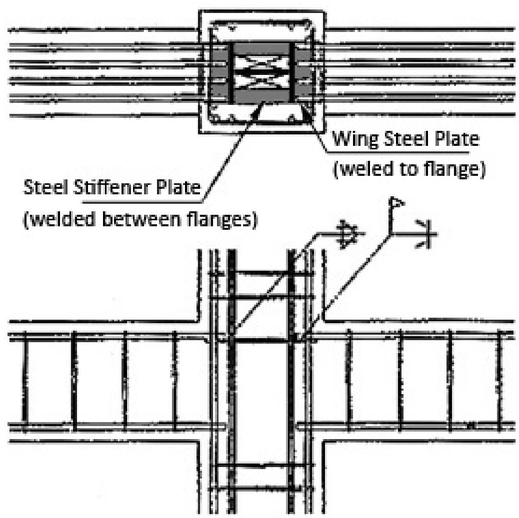
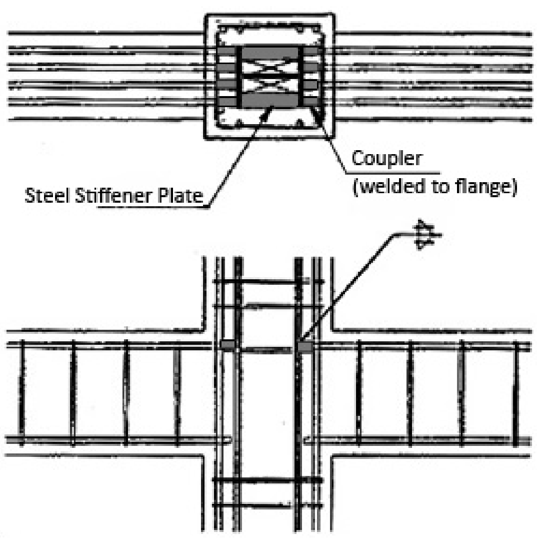


Figure 4-4: Cross-section of SRC column

Where the beam bars extend into the flange of the encased steel section, the beam bars can be anchored as follows: (1) using a wing steel plate, (2) using couplers, (3) encasing a steel H-beam in the concrete beam and welding to the column, and (4) passing the beam bars through the joint using a beam that is wider than the encased steel column (Ju and Chun 2003; Lee and Ju 2001).

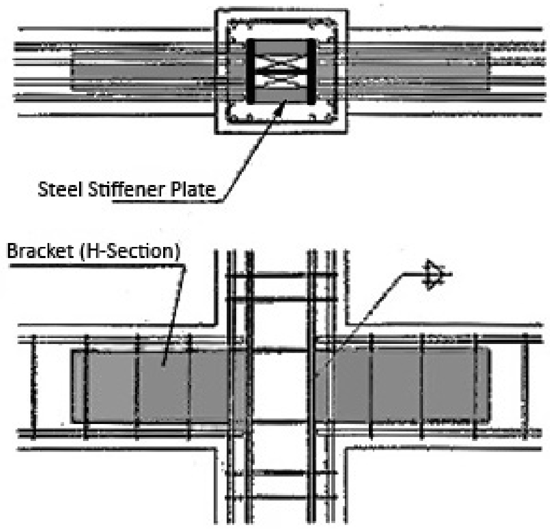


(1) Wing steel plate

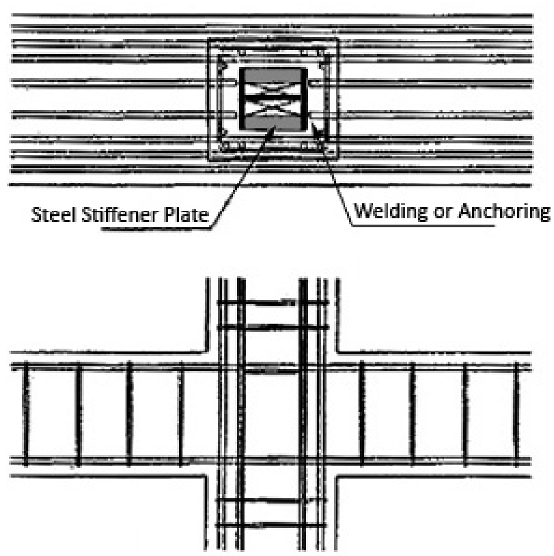


(2) Couplers

Figure 4-5: Types of anchorage in SRC column (Lee and Ju 2001)



(3) H-beam



(4) Passing bars through in a wide beam

Figure 4-5: Types of anchorage in SRC column (Lee and Ju 2001) (Cont'd)

When beam bars extend into the encased section between the flanges, the bars can be anchored by hooks within the available distance from the critical section to the web of the steel column. However, the dimension to the steel column web may sometimes be shorter than the required development length of hooked bars.

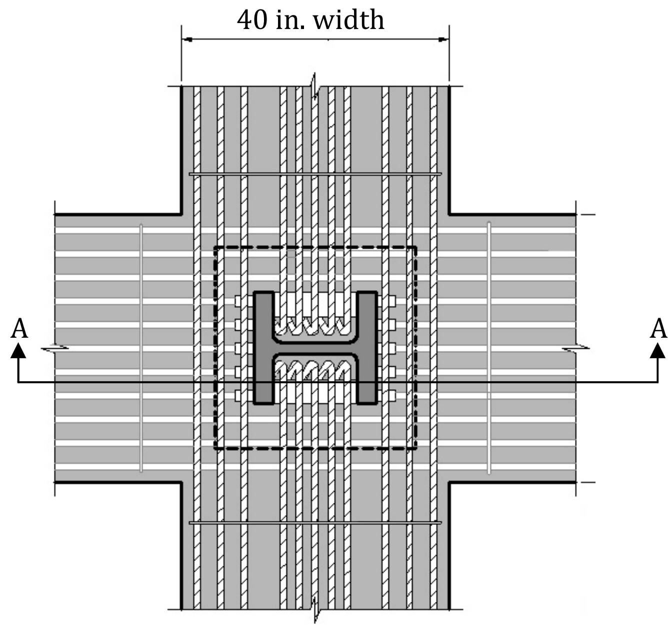
In this example, the dimensions of the members are based on the practical sizes used in Korea. The RC beam-SRC column joint is illustrated in Figure 4-6. The assumed column cross-section is 30 × 30 in. and the wide beam is 40 × 28 in. A W-shape section having the following dimensions is selected: W14 × 398, (a overall depth 18.3 in. × width 16.6 in. with a web thickness of 1.77 in. and a flange thickness of 2.85 in.).

From the previous calculation (Section 4.2.2), the required development length for a #8 hooked bar is 19 in. The available development length within the dimension of SRC column is subtracting web thickness of steel from one half of the column width: (30 in. − 1.77 in.) / 2 = 14.12 in. Thus, the development length of the hooked bar exceeds the available length to the web of the SRC column. An alternative anchorage method is required.

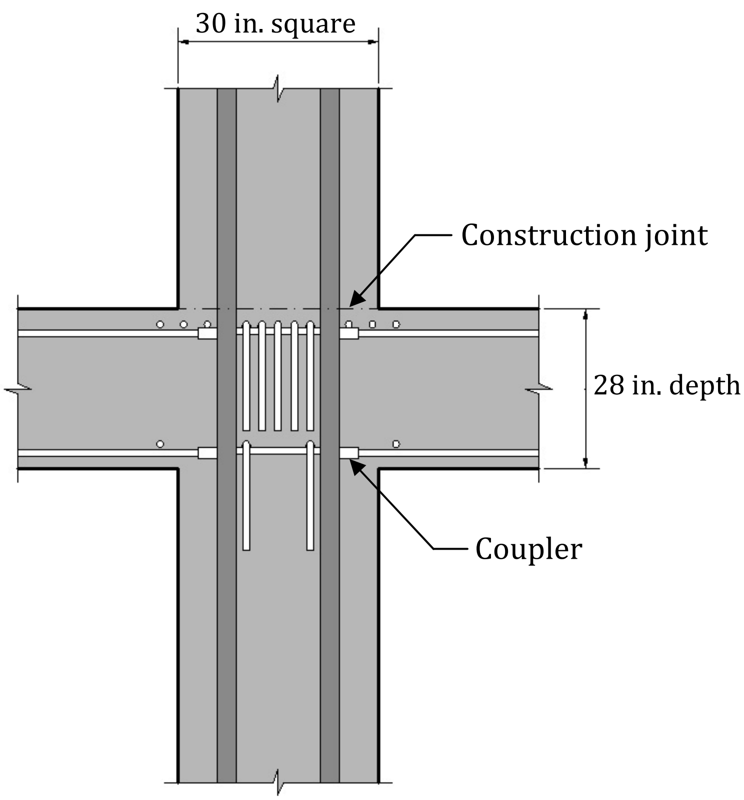
4.3.1 Assumed properties

- Specified minimum yield stress of reinforcing bar: $f_{yr} = 60$ ksi
- Specified minimum yield stress of steel section: $f_y = 50$ ksi (A992)
- Specified compressive strength of concrete: $f'_c = 4$ ksi
- Reinforcement in beam: 11-#8 bars (top reinforcement)

  4-#8 bars (bottom reinforcement)



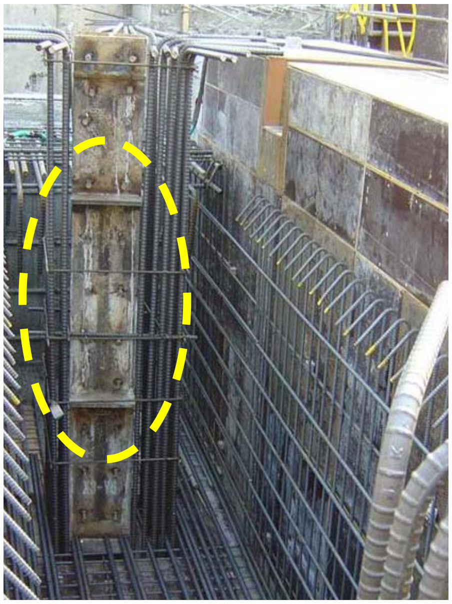
(a) Plan



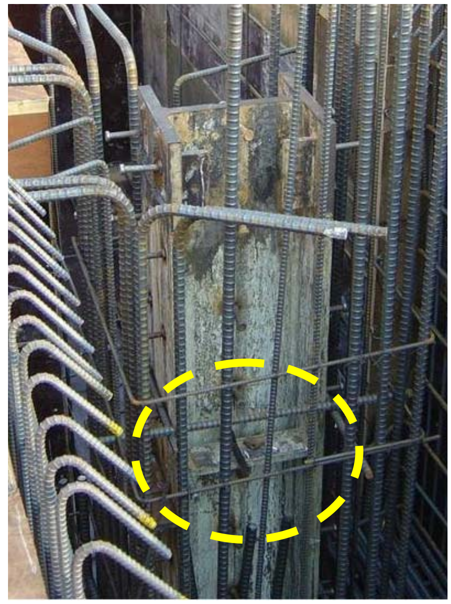
(b) A-A section

Figure 4-6: Reinforced concrete beam-SRC column joint

Figure 4-7 shows the RC beam-SRC column joint in field. For anchorage of beam reinforcement, a steel plate is welded at flange and between flanges of steel.



(a) Face of flange of steel column



(b) Face of web of steel column

Figure 4-7: Anchorage for SRC column

4.3.2 Options when short development length is required

4.3.2.1 Add more reinforcement

This method is based reducing stresses in the bars being anchored. According to ACI 318-08 Section 12.5.3 (d), l_{dh} can be reduced by the ratio of the required A_s to the provided A_s. To apply this provision, ACI 318-08 indicates that the factor for excess reinforcement should not be used in those cases where anchorage for developing f_y is specifically required to ensure ductile behavior (Section R12.5).

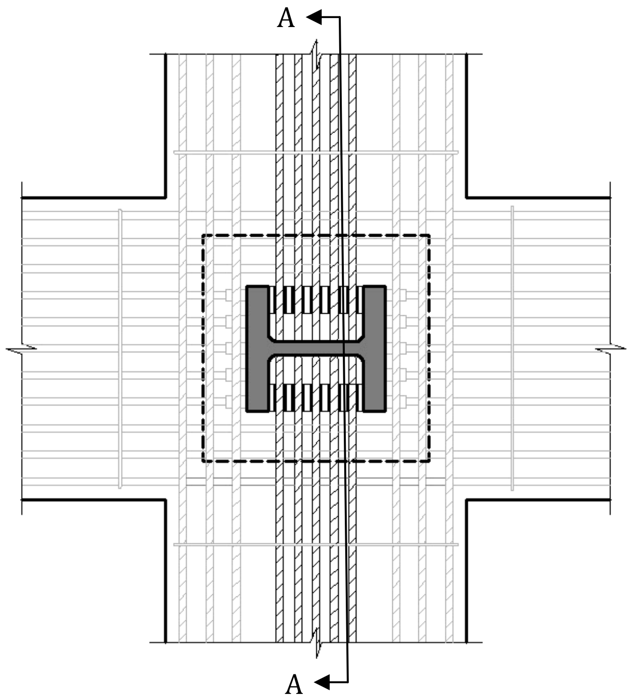
Adding more reinforcement has the advantage of reducing l_{dh}. The disadvantages conclude an increase in dead load, congestion of bars, and a less constructible joint. Thus, adding more reinforcement may influence the cost of structure and may change the geometry of the members.

4.3.2.2 Welding

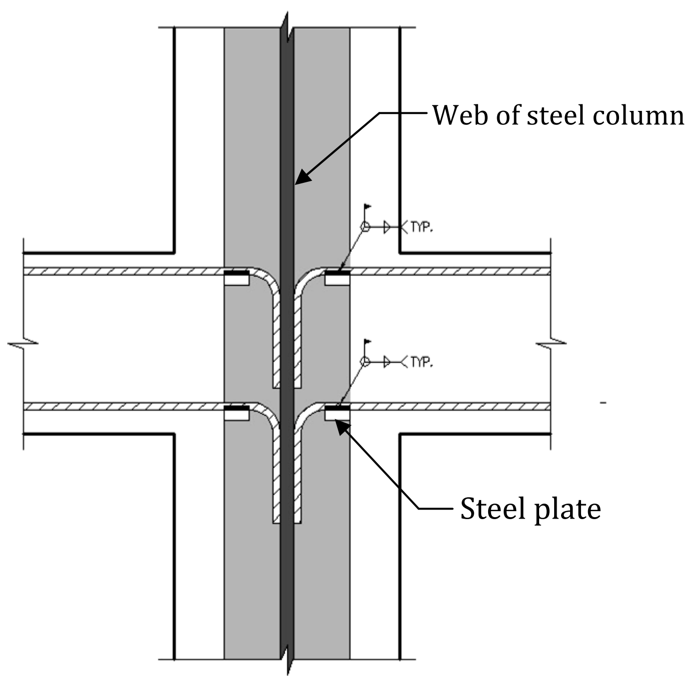
Welding reinforcement to the plate welded between flanges of the steel column provides a tensile load transfer path from the bars to encased the column. Although the tension in the bar is considered to be resisted entirely by the weld capacity, a hooked bar could increase anchorage strength because the hook also resists tension by bearing on the concrete.

In actual applications, however, welding reinforcement is restricted. ACI 318-08 specifies the use of welding on bars in Section 3.5.2 and 3.5.3. Welding of ordinary bars meeting ASTM A615 is prohibited in field. In addition, the quality of a weld must satisfy the code provisions for steel (AISC-Steel Construction Manual). The limited work space for welding adds to the difficulty in assuming quality of welding. Thus, welding should be required only if welds are careful examined. Finally, if the plates are welded in a shop, it is necessary to place plates accurately so that bars can be placed.

The detail for welding bars is shown in Figure 4-8.



(a) Plan



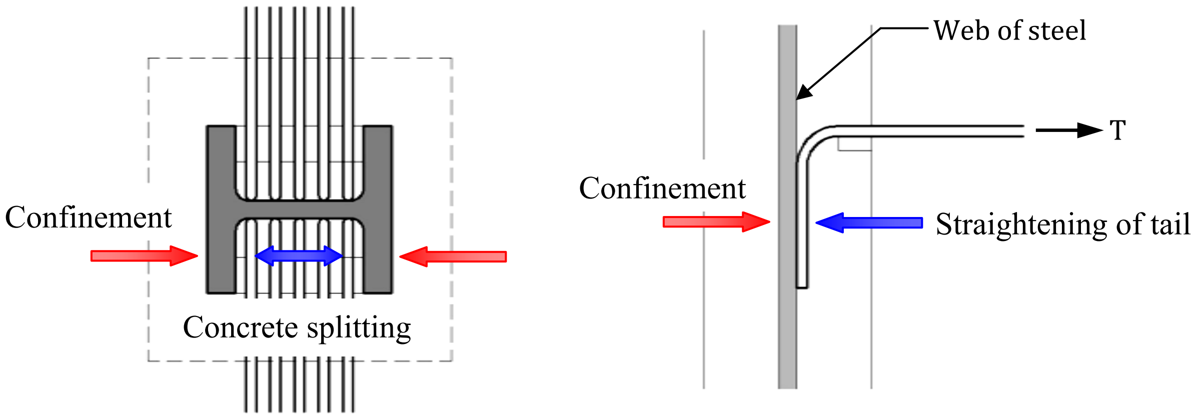
(b) A-A section

Figure 4-8: Welding reinforcing bars to the plate

4.3.2.3 Confinement check

Based on previous studies, confinement increases the bond capacity of the bars and prevents splitting failure and spalling of concrete. ACI 318-08 also states that the development length could be reduced by a factor if the reinforcement in the beam fulfills the code provision for confinement. The multiplier for standard hooks in tension, 0.7 and 0.8, described in ACI 318-08 Section 12.5.3 (a) For #11 and smaller hooks with side cover ≥ 2.5", and 90° hook with cover on bar extension beyond hook ≥ 2.5", and (b) For 90° hooks of #11 and smaller bars either enclosed within ties or stirrups perpendicular to the bar being developed, spaced $\leq 3d_b$ along l_{dh}; or enclosed within ties or stirrups parallel to the bar being developed, spaced $\leq 3d_b$ along the length of tail extension of the hook plus bend, respectively. In an SRC column, the confinement provided by the steel column and the transverse ties could reduce the development length of a hooked bar.

As shown in Figure 4-9, when the concrete within the steel column begins to split in the plane of hooked bars, splitting is restrained by the flange of the steel column. In addition, the extension of hooked bar is restrained by the web of the steel column. Thus, the multipliers by Section 12.5.3 (a) and (b) may be applicable if the steel column can provide sufficient confinement to the bars. In this example, the calculated length using the multipliers is 10.64 in. (= 0.7 × 0.8 × 19 in.). This length is shorter than the available width of SRC column, 13.94 in.; hence, 90 ° hooked bars could be used for the RC beam-SRC column joint. However, the behavior of hooked bars contained within a steel section has not been verified experimentally.



(a) By flanges of steel column (b) By web of steel column

Figure 4-9: Confinement by steel

4.3.2.4 Anchorage by plate between flanges of steel

As mentioned above, the plate between flanges of the steel column acts as a diaphragm to transfer tension from the bars welded to the column flange. Generally, the plate is located inside the bend of the bars. As shown in Figure 4-10, when the tension is applied on the bar, a reaction is developed in the opposite direction. Based on this, it is assumed that the bar tensile force is resisted by the plate at the inside of the bend.

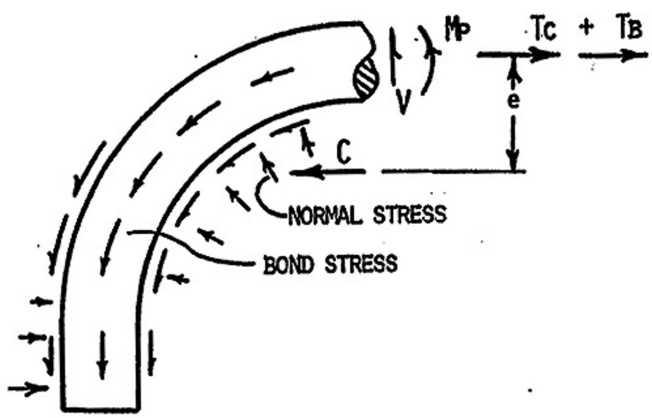


Figure 4-10: Force for hooked bar (Minor and Jirsa 1971)

According to the assumption, the following equation is used.

$$T = n \cdot A_t \cdot f_{yr} \quad \text{(Eq. 4-5)}$$

$$t = T / (B \cdot f_y \cdot 2) \quad \text{(Eq. 4-6)}$$

The factors in the equation are as follows:

n = the number of reinforcement

A_t = area of the longitudinal reinforcement in beam (in.^{2})

f_{yr} = Specified minimum yield stress of reinforcing bar = 60 ksi

f_y = Specified minimum yield stress of steel section = 50 ksi (A992)

B = width of the plate (in.)

t = thickness of the plate (in.)

Figure 4-11 shows free-body diagram of force mechanism.

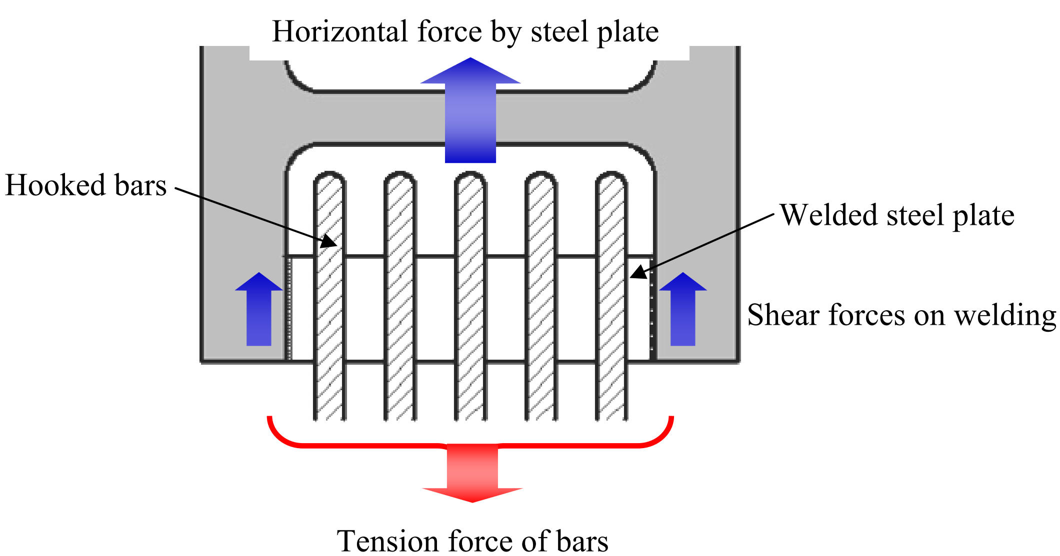


Figure 4-11: Assumed force mechanism

In this example, the width of the plate is assumed to be 3.5 in. From the structural properties,

$$T = n \cdot A_t \cdot f_{yr} = 5 \times 0.79 \text{ in}^2 \times 60 \text{ ksi} = 237 \text{ kip}$$

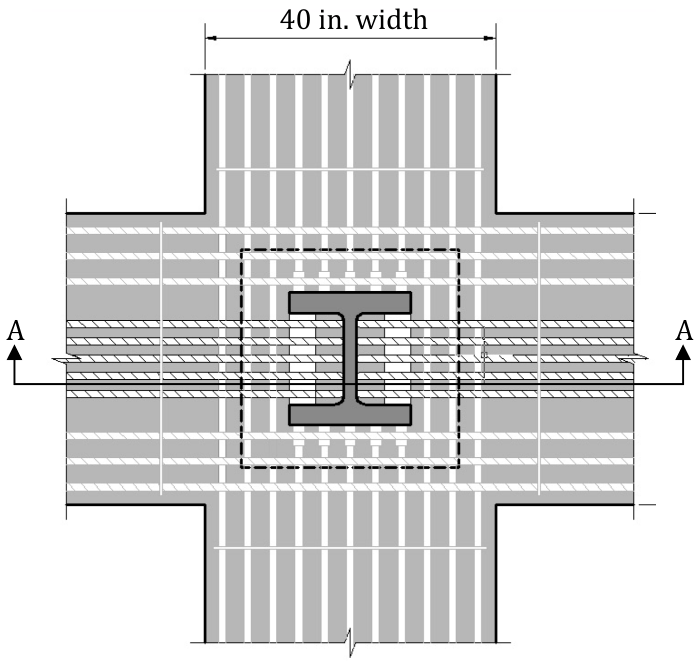
$$t = T / \left(B \cdot f_y \cdot 2\right) = 237 \text{ kip} / (3.5 \text{ in.} \times 50 \text{ ksi} \times 2) = 0.67 \text{ in.}$$

Thus, the steel plate thickness of 3/4 in. will be used.

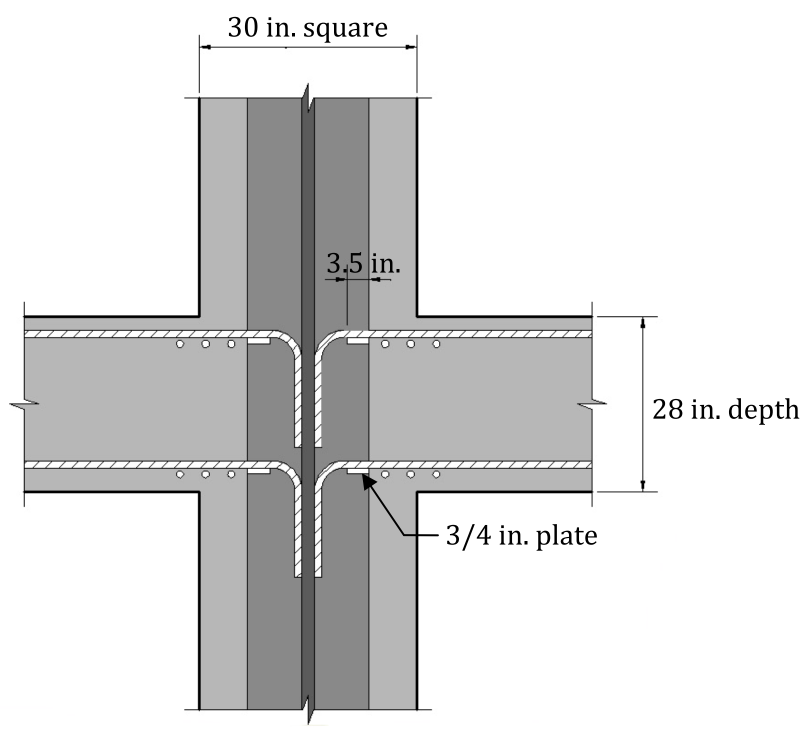
It should be noted that these are no tests available with this type of anchorage. Such tests are needed to validate the technique.

In order to place the plate, welding is needed between the steel in SRC column and the plate. Since welding is done in the field, it is necessary to carefully monitor the weld and the location of the plate.

Figure 4-12 shows the designed detail for a hooked bar anchorage in SRC column.



(a) Plan



(b) A-A section

Figure 4-12: Designed detail for hooked bar anchorage in SRC column

CHAPTER 5

Summary and Conclusions

5.1 Summary

The objective of this study was to provide an overview of hooked bar anchorages. Design examples and structural details were based on building code requirements for structural concrete (ACI 318-08) and commentary. An example of a standard hook in an exterior beam-column joint was provided, and hooked bar anchorage details for a reinforced concrete beam -steel reinforced concrete column joint were discussed.

The general behavior of hooked reinforcing bars was summarized from a review of previous studies. Then, design requirements for the development length of standard hooks were discussed and used in an example. An example of the use of hooked bar in the reinforced concrete beam-SRC column joint was provided. Some solutions of using short hooked bar development length were presented and compared.

5.2 CONCLUSIONS

Standard hooks are used when straight bar anchorage cannot be provided within available dimensions. ACI 318-08 provides standard dimensions for 90° and 180° hooks. When a tension force is applied to a hooked bar, the force is resisted by bond on the surface of the bar and by the bearing on the concrete inside the hook. For a 90° hooked bar, as the tensile force reaches the full development, the inside of hook bears on the concrete. The tail of hook straightens and compressive stresses against cover on the tail resist the prying action of the tail extension. When the hooked bar anchorage fails, crushing of the concrete inside the hook occurs. If the clear cover is not sufficient, the side cover will spall out.

In typical exterior beam-column joints, the development length of a standard hook (l_{dh}) is based on the design equation in ACI 318-08 Section 12.5.2. According to structural geometries, the multipliers as reduction factors are defined in Section 12.5.3 could be applied. The calculated development length should be placed within the dimension of members and the length of the tail should be less than the depth of the beam.

For the reinforced concrete beam-SRC column joint, the length available for anchoring a hooked bar could be less than that required due to the obstruction of the steel column. To anchor the reinforcement, other options for anchorage of bar include:

1. Adding more bars to reduce the development length (ACI 318-08 Section 12.5.3). In spite of reducing l_{dh}, there are disadvantages because the dead load on

structures may increase if wider elements are needed, congestion of bars may increase, and constructibility, especially concrete placement, may be difficul0t;

2. Welding bars to a plate between the flanges of steel is based on the concept that the tension in the bars is resisted by welding capacity. However, welding reinforcement is generally prohibited by ACI 318-08;

3. The flange and web of steel around the anchored bars could provide confinement to the hooked bar anchorages and increase the capacity of anchored bar; and

4. The plate between flanges of steel acts as a diaphragm to resist tension from the welded reinforcements and to transfer those forces to the embedded column flange. When the tension is applied on the bar, the plate placed inside the bend will help resist compression on the inside of the bend. These assumptions are no tests available with this type of anchorage. Such tests are needed to validate the technique.

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